By the Late William P. Creager and the Late Joel D. Justin

HYDROELECTRIC HANDBOOK

By the Late William P. Creager, the Late Joel D. Justin, and Julian Hinds

ENGINEERING FOR DAMS, in three volumes

I. GENERAL DESIGN
By the Late William P. Creager, the Late Joel D. Justin, and Julian Hinds

II. CONCRETE DAMS
By Julian Hinds, the Late William P. Creager, and the Late Joel D. Justin

III. EARTH, ROCK-FILL, STEEL AND TIMBER DAMS
By the Late Joel D. Justin, Julian Hinds, and the Late William P. Creager

By the Late Joel D. Justin

EARTH DAM PROJECTS

By the Late Joel D. Justin and William G. Mervine

POWER SUPPLY ECONOMICS

Engineering for Dams

By

The Late WILLIAM P. CREAGER

The Late JOEL D. JUSTIN

and

JULIAN HINDS

In Three Volumes

VOLUME I · GENERAL DESIGN

By the Late WILLIAM P. CREAGER, the Late JOEL D. JUSTIN

and JULIAN HINDS

WILEY EASTERN PRIVATE LIMITED

NEW DELHI

First Edition, 1945
Ninth Printing, 1966
First Wiley Eastern Reprint, 1968

Authorized reprint of the edition published by
John Wiley & Sons, Inc., New York, London and Sydney

This book has been published with the assistance of the
Joint Indian-American Textbook Program.

Price Rs. 15.00
Price of Original Edition, $ 9.95

Published by Anand R. Kundaji for Wiley Eastern Private Limited, J 41 South Extension 1, New Delhi 3 and Printed by Sudhir Balsaver at Usha Printers, 6 Tulloch Road, Bombay 1. Printed in India

PREFACE

During the past decade there has been a marked increase in expenditures for projects involving dams. This increase has led to an intensification of experimental research and a reexamination not only of details and methods of construction but also of many of the theories of design, all of which has resulted in a substantial improvement in the art and science of dam building. It is for this reason that the authors are presenting to the engineering profession a new work on dams which is intended to be a compendium of modern practice in sufficient detail to serve the practicing engineer as well as the student.

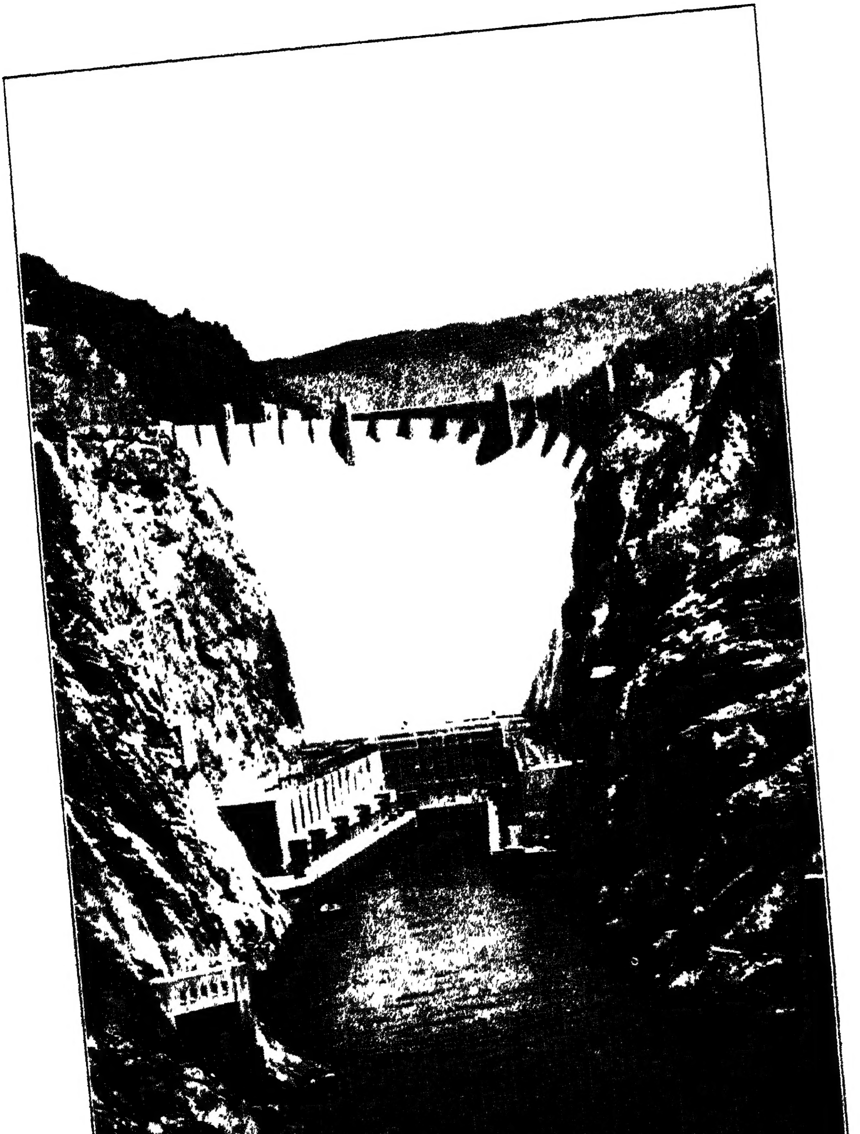
It is unfortunate that space does not permit a listing of the many engineers who have assisted the authors with suggestions, data, constructive criticisms, and much actual work. To these the authors are very grateful because, without such great help, this work would not have been possible.

Many of these persons have been mentioned in the text. Special mention should be made of Byron O. McCoy for a large amount of research work and the editing of the entire manuscript.

The authors are also indebted to the many publishers who, without exception, have given ready consent to the reproduction of illustrations from their periodicals; also to a number of government and private bodies which have been unstinting in their aid.

William P. Creager.
Joel D. Justin.
Julian Hinds.

March, 1944.



Boulder Dam, the highest in the world (726 feet).

CONTENTS

In this book article headings are listed in the table of contents only in the volume in which they occur, but all chapter titles for the three volumes are listed in each volume. A complete index covering all three volumes appears in each volume.

Volume I

CHAPTER 1. INVESTIGATION OF DAM SITES

CHAPTER 2. THE CHOICE OF TYPE OF DAM

CHAPTER 3. PREPARATION AND PROTECTION OF THE FOUNDATION

I. GENERAL

II. TREATMENT OF ROCK FOUNDATIONS

III. TREATMENT OF EARTH FOUNDATIONS

IV. CONTROL OF EROSION BELOW SPILLWAYS

V. BIBLIOGRAPHY

CHAPTER 4. HYDRAULIC MODEL STUDIES

CHAPTER 5. FLOOD FLOWS

I. FLOOD PEAKS

II. FLOOD HYDROGRAPHS

A. Basic Hydrologic Analysis

B. Rainfall Analysis

C. Infiltration

D. Regimen of Runoff

E. Unit Hydrographs

F. Spillway Design Storms

Volume II

Volume III

CHAPTER 1

INVESTIGATION OF DAM SITES

1. General. Thorough investigation to determine the most desirable and economic site for a dam should precede the designing and constructing of the dam. Such an investigation will include surveys, topographic mapping, geologic studies, and subsurface investigations. Also tests will have to be made on the materials in the foundations and on the materials of which the dam may be composed, and the results of the tests will have to be studied and their significance realized. For this subject the reader is referred to Chapter 16.

2. Advisable Extent of Investigations. The advisable extent of such investigations depends in part on the magnitude of the project and in part on how obvious the subsurface conditions are. Thus the amount of time and money devoted to investigation of a site where the entire foundation consists of solid granite, water worn and exposed to view, would probably be much less than that required at a site in a valley having a deep accumulation of overburden on stratified and folded rock of unknown quality. Similarly, it is seldom necessary to make as extensive an investigation for a low diversion dam as for a high earth or masonry dam.

Occasionally the investigations of dam sites have been overly thorough and expensive. On the other hand it is frequently difficult to get an owner to stand for a sufficiently thorough subsurface investigation prior to construction. Consequently trouble and excessive cost have often resulted from insufficient preliminary investigation.

3. Reconnaissance. Serious investigating should be preceded by considerable reconnaissance work and rough figuring. The engineer when he first goes on the ground should have some idea of the sort of site or sites for which he is looking. If for a power dam, how much head is desired? If for water-supply, or storage, or flood-control dam, what capacity in acre-feet is desired?

At the time of the reconnaissance the engineer should know the approximate spillway requirements for the various sites because it sometimes happens that an unusually narrow gorge is not by any means the most desirable site because it may be too short to provide economically for the required spillway capacity.

The reconnaissance should involve visiting all possible sites which are available for consideration and gathering such information as is obtainable without subsurface exploration relative to such sites. The best maps available should be obtained. In many sections the Government topographic maps, made by the United States Geological Survey, are extremely useful to engineers on reconnaissance work. Aneroid barometers, hand levels, a metallic tape, and a kodak are usually desirable equipment.

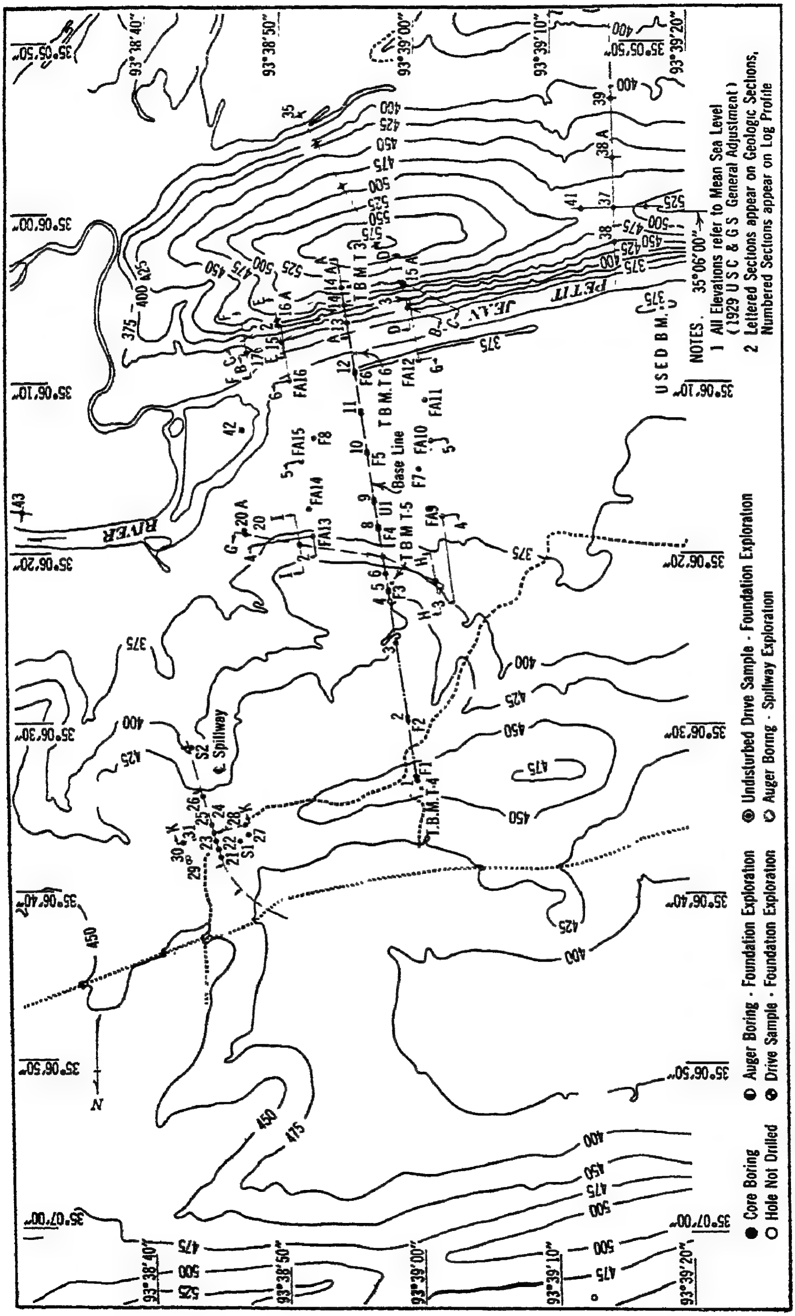


FIG. 1. Topography of Blue Mountain dam site, Petit Jean River, Arkansas. (*Courtesy of U. S. Engineer Office, Little Rock, Ark.*)

It is obvious that a reconnaissance should include all available sites which have any possibility of being utilized. A relatively small amount of office study with the data gathered in the reconnaissance will usually result in the elimination of all but a few of the sites.

4. Preliminary Investigation. The preliminary investigation usually requires:

(*a*) A not too precise stadia site survey with the resulting topographic site map.

(*b*) Some investigation of the overburden.

(*c*) A few borings, say from 6 to 50, according to the magnitude of the project and the character of the foundation.

(*d*) A preliminary geologic investigation and report.

(*e*) Investigation of available construction materials, such as earth and gravel and concrete aggregates.

(*f*) The determination of public utilities which the dam might affect, such as roads, bridges, railroads, telephone and telegraph lines, pipe lines, and power plants.

(*g*) In the relocation of the above facilities a fairly accurate topographic map of the basin is essential.

(*h*) Hydrologic studies.

(*i*) The checking of high-water marks and their use in determining spillway capacity requirements. (See Art. 8, Chapter 5.)

The objective of the preliminary investigation is to obtain only sufficiently precise data to permit office studies and estimates of cost of sufficient accuracy to determine the most economical and suitable site among the several selected by the reconnaissance survey.

A consideration which, for storage dams, affects the choice of general location is the quantity of silt carried by the stream. In some streams this is enormous and may in the course of a few years sufficiently fill the reservoir to destroy its usefulness for storage.

Sluices in the dam are never effective in preventing the silting of the reservoir except near the dam. In slow-moving heavily silt-laden streams in flat valleys, the silt will deposit not only in the pond created by the dam, but in the river bed above the pond, causing a rise in the river bottom and in the water surface for several miles above the upper end of the reservoir. The amount of silt carried by rivers ranges from an insignificant amount up to 7000 parts per million or more. The average silt carried by the Mississippi River at St. Louis, for instance, is about 1500 parts per million; i.e., for each liter of water, 15 mg of silt (dry weight) or for each pound of water 0.0015 lb of silt.

The 1938 flood in southern California brought down deposits whose maxima were 60,000 cu yd per square mile from 160 sq miles, 70,000 from 50 sq miles, and as much as 120,000 cu yd per sq mile from 0.25 sq mile.

5. Final Investigation. After the preliminary investigations at the several sites have been made and office studies and estimates for each of them completed, one of the several sites is selected for final, precise investigation. The

site survey and the resulting topographic maps should be sufficiently accurate and precise to serve all the purposes of construction. All necessary borings, test pits, subsurface explorations, geologic studies, and tests on the materials in the foundation and in the proposed borrow pits will be made.

As a result of the final investigation the engineer should have available all the pertinent data to proceed with the detailed designs of the structures and the making of a control estimate of cost for construction.

The line of demarkation between preliminary and final investigation of a site is not sharp. One often blends into the other. The point is that in the early stages of investigation where several sites are involved, the amount of investigation should be limited to that necessary to determine the relative merits of the sites, thus avoiding the possibility of making a precise and costly investigation only to find that subsurface conditions are such that the site will have to be abandoned in favor of one of the alternative sites.

The final investigations are usually supervised by the engineer who has conducted the preliminary examinations or are at least conducted in accordance with his recommendations. The principal items are:

(*a*) To determine the relative merits of two or more sites for the dam in question so that a final location can be adopted.

(*b*) To determine the type of dam to be used.

(*c*) To settle beyond a doubt, by subsurface investigations, the nature of the foundation as affecting the safety and cost of the dam.

(*d*) To fix the limits of the lands to be controlled for flowage, for the sites of structures, and for other necessary purposes.

(*e*) To determine the extent and character of relocation of railroads and public highways necessary on account of raising the water surface.

(*f*) To ascertain the character of the Government regulations to be observed.

(*g*) To obtain sufficient information for an accurate estimate of cost.

(*h*) To determine the final location of the dam, construction equipment, camps, cofferdams, construction highways and railroads, as well as the probable source of materials of construction and all other information needful to the constructing engineer.

(*i*) To obtain all necessary information affecting the design of the dam.

6. Choice of Location. A dam is frequently a unit in a more or less extensive project involving a number of structures of various types. The general location, therefore, is fixed by factors related to the purpose of the project and is affected by considerations but remotely allied to the text of this book. Bearing in mind the fact that the general location to be adopted is that which, at reasonable cost, will be best suited to the purpose for which the dam is intended, we have only to consider here those factors which affect the cost and safety of the dam, and the choice of its exact position.

The general location having been chosen, the exact position will be fixed after careful consideration of each of the following factors:

(*a*) The character of the foundation.

(*b*) The topography of the earth and rock surfaces at the site and its effect

on the dimensions of the dam, quantity of material to be excavated, and other factors.

(c) Availability and character of materials for construction.

(d) The value of the necessary lands and water rights.

(e) Requirements as to coffers, pumping, conduits, and other provisions necessary for unwatering the site.

(f) Transportation facilities and the accessibility of the site.

(g) Availability of suitable sites for construction equipment and camps.

(h) The safety of the structure.

The foundation is one of the most important factors in the final location. It should be sufficiently impervious for the use intended or capable of being made so and have sufficient strength to properly sustain the weight of the dam and the pressure of the impounded water.

It is desirable that the valley should have a width which is adequate but no greater than that required for the dam, including its spillway, powerhouse, navigation lock or other necessary appurtenance. Except an earth or rock-fill dam, a part of the dam is usually designed to act as a spillway. However, for an earth or rock-fill dam, the spillway must be a separate structure and a site for it must be located.

The location, quality, and cost of local materials for construction purposes, including concrete ingredients, pervious and impervious soil for earth-fill dams, rock for rock-fill dams, and riprap, influence the choice of location of the dam.

7. Topography, Aeroplane Mapping.[1] Without topographic relief there would be no use for dams. So topography is of prime importance in any investigation of a dam site. In recent years the several methods of aeroplane mapping with ground control have been slowly superseding the older stadia methods for producing small-scale topographic maps.

Under present methods topographic maps of a scale which will permit the use of contours of 10-ft intervals or greater are made with plotting equipment utilizing stereoscopic pairs of aerial photographs. The most common type of this equipment in use is that which was originally manufactured by the Zeiss Aero Topograph Company under the trade name of Multiplex. The equipment has later been produced domestically by the Bausch and Lomb Optical Company and has been adopted by the Corps of Engineers, United States Army, as standard mapping equipment in both its military and civil phases of topographic mapping.

The method of procedure in the preparation of a topographic map by use of the multiplex equipment consists of (1) the taking of aerial photographs of the area to be mapped; (2) the establishment of sufficient ground control or survey control to permit the adjustment of the spatial models[2] to the accuracy re-

[1] Data in Arts. 7 to 14, inclusive, furnished by Major T. F. Kern, District Engineer, Corps of Engineers, Little Rock, Arkansas, and prepared by Francis P. Newman, Head of Surveys and Mapping Section.

[2] The term *spatial model* refers to the model of the earth's surface formed in space by the intersection of the projected image rays of light when viewed with suitable filter spectacles.

quired in the map; (3) the multiplex plotting operation, which forms and transposes the spatial model into the actual map; and (4) the drafting and reproduction process.

8. Aero-Projection Method. The multiplex or aero-projection method utilizes combinations of pairs of the instruments from which are projected in turn positive images of successive pairs of aerial photographs. A spatial model of the landscape is produced by reversal of the photographic process and projected with rays of two complementary colors which are brought to intersection in space by suitable adjustment of the instrument. This plastic model is viewed through spectacles containing filter glass of corresponding complementary colors and is scanned and measured with the aid of a movable and adjustable measuring mark. A pencil centered beneath the measuring mark permits the model to be produced on the plane in true orthographic projection by the continuous tracing of its planimetry and elevation.

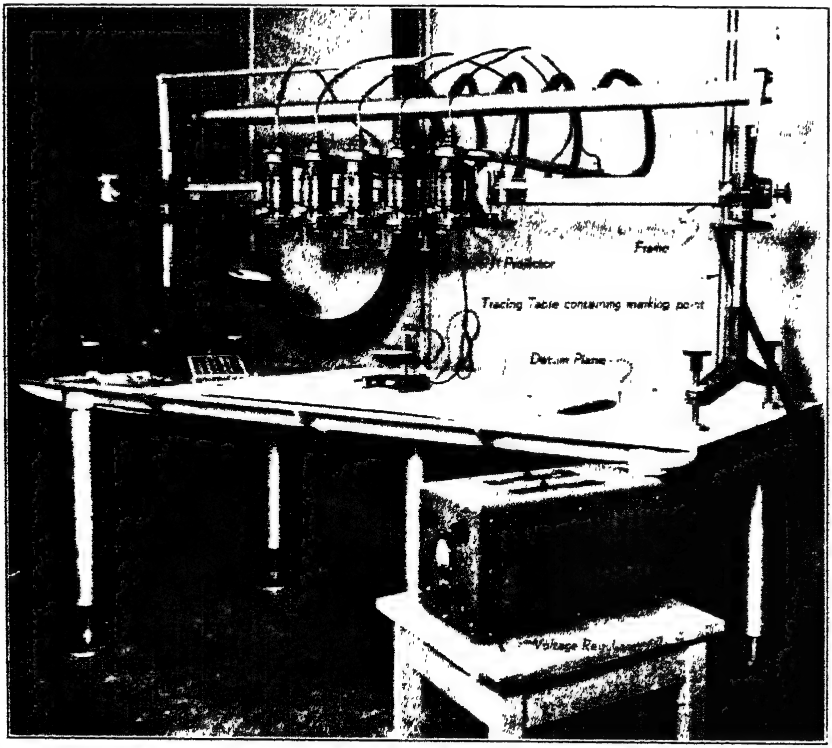


FIG. 2. Typical multiplex set-up for topographic mapping from aerial photographs. (*Courtesy of U. S. Engineer Office, Little Rock, Ark.*)

9. Multiplex Equipment. The multiplex equipment consists of a plane surface table of rugged construction which becomes the datum plane, a bar mounted on a frame so that its parallelism to and distance from the plane sur-

face can be easily regulated, projectors or miniature cameras, a small circular table carrying the measuring mark, a reduction printer which reduces in proper ratio the aerial negative to the glass positive which is to be projected, and other pertinent accessories.

Fig. 2 shows a typical multiplex set-up for topographic mapping from aerial photographs.

10. Ground Control for Multiplex Mapping. Ground control to which the multiplex spatial model is to be oriented is obtained by survey methods of an order of accuracy sufficient to insure the accuracy required in the finished map. In general, to obtain 10-ft accuracy at the maximum altitude to which photography may be obtained, horizontal control of at least third-order accuracy is required across the lines of flight at intervals of about 6 miles. Vertical control of fourth-order accuracy is necessary in sufficient quantity to determine at least four elevation points for each stereoscopic pair. To obtain the required accuracy this vertical control must of necessity be tied to levels of a higher order, which for economy are generally along highways, railroads, or most accessible routes.

11. Multiplex Plotting. In the multiplex plotting operation diapositives [3] are properly oriented in two projectors so that they assume the position of the aerial negative in the taking camera. The spatial model is formed by recovering the taking camera's position during successive exposures, and the model is oriented to the ground control plotted upon the drawing medium. The operator then transfers, by means of a pencil centered under the marking point, the planimetry and topography of the model to the drawing medium. This is done by manipulation of the marking point, which is a small dot of light, in the center of a round white disk, capable of being moved and measured in a vertical direction and moved manually in a horizontal position so as to trace the desired planimetry or contours in an orthographic projection. This required information is obtained by keeping the small dot of light in contact with the surface of the spatial model,[4] the elevation being determined by the ratio of the measured vertical movement of the dot of light to the scale at which the mapping is being performed. After the multiplex operation is completed the resulting sheet is carefully edited from the proper interpretation of the photographs, and is then prepared for the desired method of reproduction.

12. Comparative Cost of Multiplex Mapping. The cost per square mile of area contoured by multiplex will vary in direct ratio to the usable portion of the spatial model. This usable portion in the mapping of reservoir areas varies in accordance with the terrain. In shallow disklike reservoirs it will approach a maximum, whereas in the long narrow finger-type reservoir it will approach a minimum. Only in contouring the entire area covered by the aerial photography can the maximum saving in cost by use of the multiplex method be realized.

[3] The term *diapositive* refers to a small positive aerial photograph printed on emulsion-coated glass so as to permit its projection by artificial light.

[4] See footnote 2.

The total cost of contour mapping by use of the multiplex system will vary materially with the field work which has to be done and the amount of cultural detail found within the area, but the biggest variable is the percentage of total area photographed which is utilized for contouring.

The following tabulation gives the usual range of total unit costs for various percentages of total area photographed which is contoured, i.e., cost is per square mile contoured, but area photographed is usually several times that area.

Per Cent of Total Area Photographed Which is Contoured	Usual Range in Total Cost Per Square Mile Contoured
25	$300 to $400
30	235 to 315
35	200 to 270
40	165 to 220

The unit cost includes flying, photographing, ground-work control, mapping, etc., and overhead. Under exceptional conditions the range in cost may be much wider than here indicated.

In order to make comparison between multiplex and plane-table methods of preparing topographic maps it would be well to review the modern plane-table method which includes the use of aerial photographs. From a network of vertical and horizontal ground control plotted upon the photographs, the contours are determined. These photographs are then returned to the office where a radial line plot[5] is made for proper scale and orientation. This radial plot is then transferred to a base sheet, upon which the detail from the photographs is delineated with the use of a pantograph or optical projector. The sheet is then inked and finished for reproduction. The plane-table party generally consists of four men. In multiplex operation one multiplex operator can be considered as replacing the above four-man party for the same period of time. Since aerial photographs are used in both procedures, the flying cost is the same. The ground control required for both operations is approximately the same, and after the completion of the base sheet by either method, the editing, drafting, and reproduction costs are the same. The principal advantage of the multiplex over the plane table, other than the saving of cost for personnel, is that the multiplex may be operated two or even three shifts per day regardless of weather conditions. Based upon the best comparisons available, it is indicated that a saving of 30 to 40 per cent in both time and cost may be affected through use of the multiplex system.

13. Utilization of Aeroplane Topographic Maps. Such aeroplane topographic maps giving whole river valleys and basins for many miles are fre-

[5] A *radial line plot* is a method of removing from selected image points on an aerial photograph the displacement and distortion due to tip, tilt, varying scale, and difference in relief. For a detailed description of the theory and use of the radial line plot the reader is referred to Technical Manual No. 5-230, "Topographic Drafting," which may be obtained from the Superintendent of Documents, Washington, D. C.

quently extremely useful in connection with ssance work and preliminary investigations. Such maps bring ou s and conditions which one might miss even though he walked the e er bank for the section under consideration.

Aeroplane maps even without contours are ly useful, as they show up all topographic features, fence lines, wood plowed lands, structures and public utilities. A recent development Anaglyph."[6] The actual photographic map of this type looks like a mear at first glance. To look at it one uses a "Macyscope" (left len d right lens blue). This brings out the relief, and many features not n the ground or even from an aeroplane flight over the terrain are brou . Thus, one may visually compare the relative topographic advantage rnative dam sites and may discover unsuspected possible locations for spillways.

14. Site Maps. An aeroplane topograph may be sufficiently precise to use for the site map for preliminary inv ions, but for final investigations the topographic maps should be prod stadia or plane-table methods with a considerable amount of chaining done for important base lines. A coordinate system should be laid on the topographic map of the final site and all borings, test pits, and stru actual and proposed, should be referred to it throughout the investigat d the construction. The contour interval should usually be 2 ft or 5 ording to the steepness of the country; and the scale is usually 100 feet s to the inch. (See Fig. 1.)

15. Geologic Investigations. For the i igation of dam sites the use of a geologist experienced in the study of dam is essential. All good geologists

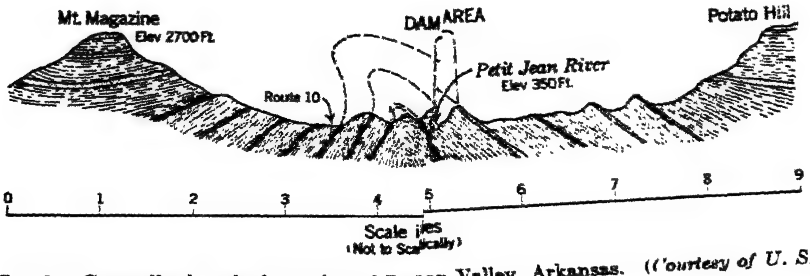


FIG. 3. Generalized geologic section of Petean Valley, Arkansas. (*Courtesy of U. S. Engineer Office, le Rock, Ark.*)

have extensive imaginations and the gineer never ceases to marvel at the completeness of the geologic report, ich a geologist is often able to prepare after a few days of scouting around dam site. The data may not be very precise but will be extremely helpful indicating where the drilling had best be done. (See Fig. 3.)

[6] American Colortype Co. An anaglyp is a picture resulting from the printing of two images nearly in superposition, one in e-green and the other in red, which will give a stereoscopic effect when viewed throug a pair of complementary colored glasses.

quently extremely useful in connection with reconnaissance work and preliminary investigations. Such maps bring out features and conditions which one might miss even though he walked the entire river bank for the section under consideration.

Aeroplane maps even without contours are extremely useful, as they show up all topographic features, fence lines, wooded and plowed lands, structures and public utilities. A recent development is the "Anaglyph."[6] The actual photographic map of this type looks like a brown smear at first glance. To look at it one uses a "Macyscope" (left lens red and right lens blue). This brings out the relief, and many features not noted on the ground or even from an aeroplane flight over the terrain are brought out. Thus, one may visually compare the relative topographic advantages of alternative dam sites and may discover unsuspected possible locations for saddle spillways.

14. Site Maps. An aeroplane topographic map may be sufficiently precise to use for the site map for preliminary investigations, but for final investigations the topographic maps should be produced by stadia or plane-table methods with a considerable amount of chaining being done for important base lines. A coordinate system should be laid down on the topographic map of the final site and all borings, test pits, and structures, actual and proposed, should be referred to it throughout the investigation and the construction. The contour interval should usually be 2 ft or 5 ft according to the steepness of the country; and the scale is usually 100 feet or less to the inch. (See Fig. 1.)

15. Geologic Investigations. For the investigation of dam sites the use of a geologist experienced in the study of dam sites is essential. All good geologists

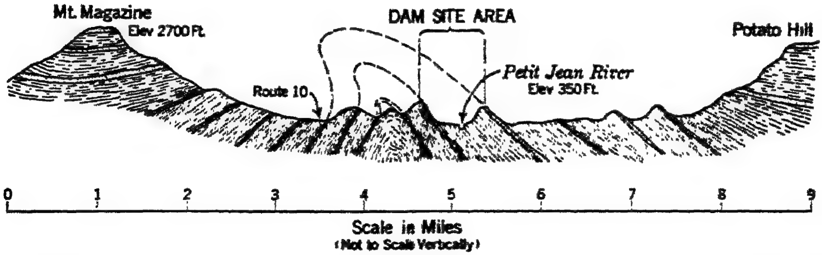


Fig. 3. Generalized geologic section of Petit Jean Valley, Arkansas. (*Courtesy of U. S. Engineer Office, Little Rock, Ark.*)

have extensive imaginations and the engineer never ceases to marvel at the completeness of the geologic report, which a geologist is often able to prepare after a few days of scouting around a dam site. The data may not be very precise but will be extremely helpful in indicating where the drilling had best be done. (See Fig. 3.)

[6] American Colortype Co. An anaglyph is a picture resulting from the printing of two images nearly in superposition, one in blue-green and the other in red, which will give a stereoscopic effect when viewed through a pair of complementary colored glasses.

In fact it should be an invariable rule never to start a drilling program in work of importance without the advice of an experienced dam geologist. Slides, faults, folds, sink holes, dips, and strikes may carry implications to a geologist which entirely escape the engineer. For instance, a limestone which is interbedded with strata of shale and which has not been subject to extensive fracturing or faulting is much less apt to have extensive solution channels or cavities than one which has such fractures and does not have the interbedding of shales. Furthermore some rocks are much less subject to solution than others.

Some sites have been extensively and expensively prospected with diamond drilling only to find unsatisfactory foundation conditions, where if a good dam

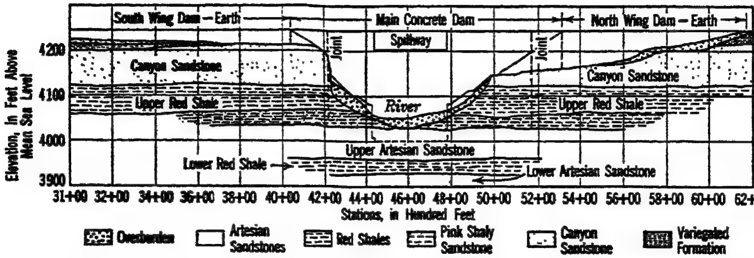


FIG. 4. Geologic section across Canyon Conchas Dam, New Mexico. (From "Engineering Geology Problems at Conchas Dam, New Mexico," by Irving B. Crosby, *Trans. Am. Soc. Civil Engrs.*, Vol. 105, 1940, p. 583.)

geologist had been engaged beforehand, he could have told the engineer in advance approximately what he would find and could have directed him to another site where prospects of success were more propitious. In the interpretation of the cores from the borings, the geologist can also be of material assistance. On work of magnitude it is quite usual to utilize the services of a geologist throughout the period of investigation and also to engage the services of a particularly experienced dam geologist as a consultant.

As the work of subsurface exploration continues geologic sections at and near the site are drawn up with the help of the geologist. Such geologic sections are very useful to the engineer in studying his foundation problems and his grouting program. In Fig. 4 is shown a geologic section at the site of Conchas Dam, New Mexico, and Fig. 5 shows a geologic section at the site of Gatun Dam, Panama. Geologic sections greatly facilitate the study of foundation conditions. They should never be used as information for bidders or as a part of the contract plans. This is for the reason that they necessarily involve the use of imagination and the engineer may thus lay himself open to the charge of misrepresentation. Only the cores of the borings themselves and the carefully interpreted log of the holes should be utilized as information for bidders.

16. Subsurface Exploration. When a dam is to be built it is usually necessary or desirable to conduct extensive subsurface explorations in order to aid

the geologist, the soils engineer, and the project engineer to determine the following factors:

(*a*) Suitable and economic type of dam to construct.

(*b*) Quality of materials in overburden if an earth dam is to be utilized. (1) Are the overburden materials of sufficient shear strength so that an earth dam can be constructed with slopes determined by the materials in the dam or is the shear strength of the foundation material enough lower that it becomes the controlling factor in determining the slopes of the earth dam? (2) Is the material in the overburden sufficiently impervious or will it be necessary to construct a cutoff to ledge rock or an upstream blanket in order to assure safety against piping or to prevent the loss of uneconomic quantities of water? (3) Does the overburden contain soluble material in sufficient percentages to be hazardous? (4) Are there borrow pits of suitable material available nearby for constructing the dam, and what is their depth and extent? (5) Does the underlying ledge rock contain caverns, joint planes, and solution channels which would necessitate treatment even if an earth dam is utilized?

(*c*) Quality of the ledge rock if a concrete dam is to be built. (1) What is depth to which rock is weathered to such an extent that it will have to be removed in order to form a suitable foundation for a concrete dam? (2) Are there seams and joint planes which will require extensive grouting, and to what depth? (3) Is the foundation ledge badly shattered or does it contain exten-

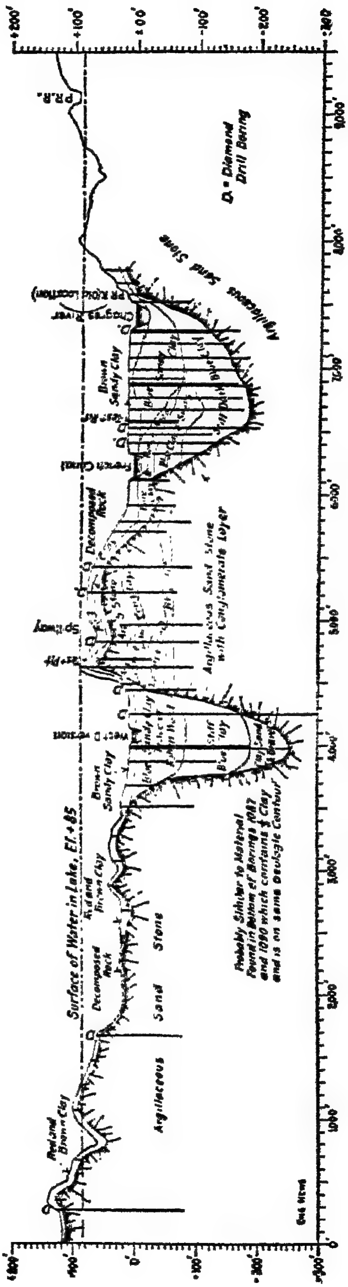


FIG. 5. Geologic section on axis of Gatun Dam.

sive, unhealed faults which may be the source of structural weakness or excessive leakage? (4) The strength of the rock, its hardness and durability. (5) The availability of suitable concrete aggregate. Does the river bed or its vicinity contain deposits of sand and gravel which will pass the usual standard tests? (6) Are conveniently located quarry sites available in ledge rock which will pass the usual standard tests for concrete aggregate? (7) Does the foundation ledge rock contain cavities or solution channels which may require extensive treatment?

17. Methods of Subsurface Exploration. By far the most satisfactory way to explore the underground is to dig test pits, shafts, and tunnels. Get down there, examine the material, take samples and test them. Ordinarily, if such methods were depended on exclusively, the cost of thorough subsurface exploration would frequently be prohibitive. Consequently it is necessary to rely on methods which are supplementary or alternative, such as electrical prospecting, seismic exploration, and borings of various kinds.

18. Electrical Resistivity and Seismic Methods of Prospecting. In some cases the engineer is not greatly interested in the nature of the overburden above the ledge. What he wants to know is how far it is to rock and what the character of the rock is when it is reached. In such cases after a few widely scattered boreholes have been placed to serve as a basis for reference either the electrical or seismic method of prospecting may be used to advantage. Any one who thinks that these methods can be used in lieu of diamond drilling may receive a bitter disappointment.

The methods are supplementary and can be used to develop general surface features of the ledge and thus indicate where it is advisable to place additional holes through the overburden and diamond-drill holes into the rock. If used in this manner with a full understanding of their limitations, they may lead both to a material economy in the cost of subsurface exploration and a better knowledge of subsurface conditions.

The electrical-resistivity method is dependent on the difference in conductivity of the water contained in the ledge rock.

The seismic method requires the measurement of the rates of propagation of waves caused by explosions and is dependent on the difference in the elastic properties of the ledge rock and the overburden. Both of these methods require the use of trained and experienced personnel. For most engineers interested in foundation work it is sufficient to know the conditions under which the methods might be useful. Then if he decides that one or both of these methods would be useful to him, he should engage a specialist to make the exploration. Under some conditions one method is superior to the other and consequently there is an advantage in engaging a specialist who is familiar with both methods.[7]

[7] See I. B. CROSBY and E. G. LEONARDON, "Electrical Prospecting Applied to Dam Sites," Technical Publication No. 131 (Class L, Geophysical Prospecting No. 7); also W. J. MEAD, "Engineering Geology of Dam Sites," Second Congress on Large Dams, Washington, D. C., 1936.

19. Test Pits. Test pits are among the most desirable methods of exploring the overburden. They permit the engineer to actually get down there and see what the overburden looks like in its undisturbed condition. Test pits and test trenches also provide the best opportunity for obtaining undisturbed

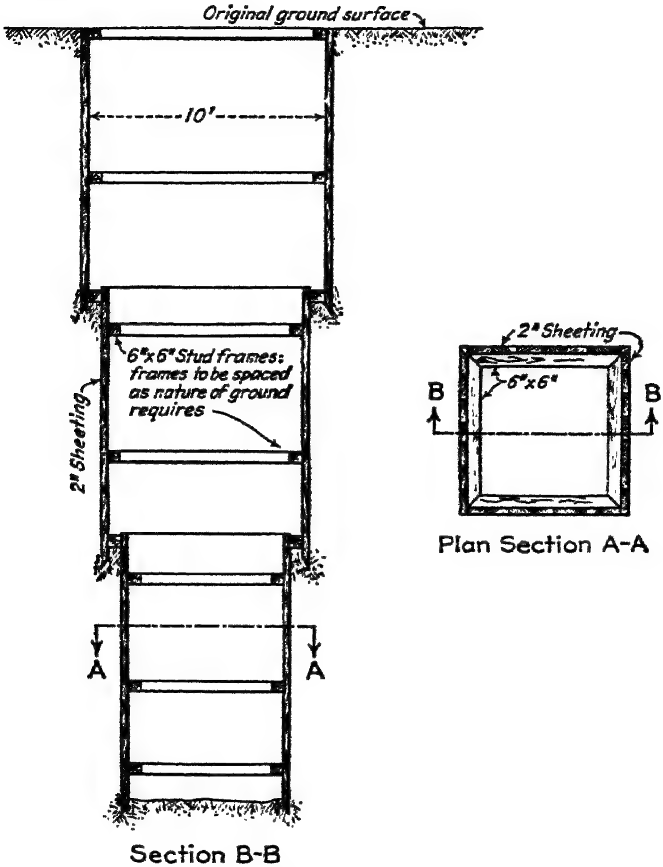


FIG. 6. Test pit showing arrangement of timbering and sheeting. Note method of pointing sheeting to prevent crowding in of pit.

samples, which are essential if the true strength and other characteristics of the material are to be learned. One would think that the sinking of a test pit was so simple that any one should be able to do it. However, the observance of the many mistakes, such as pointing the sheeting on the wrong side, improper bracing, etc., would seem to make it desirable to say a few words about sinking test pits.

In sinking a test pit of considerable depth, care should be taken to start the pit large enough at the top. The sheeting should run in a vertical direction and

be driven down as the depth of the pit increases. The studding should be made up of horizontal frames to retain the sheeting. It is useless to try to drive sheeting less than 2 in. thick. Two-by-sixes or two-by-eights 12 to 16 ft long should be used. They should be pointed to wedge shape at the lower end. The adzing for this pointing should be done on one side of the plank only, and the point when set up should be on the outside of the pit, so that the marks of the adz are exposed to view from inside the pit. This is a small detail, but important, as, if reversed or center pointed, the sheeting will crowd in at the bottom. When one set of sheeting is driven to its full depth, the next set must be stepped in back of the studding frame already in place before driving can be started. For the studding four-by-sixes for a hole of ordinary depth in ordinary material will generally be sufficient. The spacing of the studding frames should be as the nature of the ground requires. The pit should be square in plan and its dimensions should depend upon the maximum depth which it is desired to reach. The dimensions at the bottom should not be less than 4 by 4 ft, and 2 ft each way should be allowed for each additional set of sheeting. Thus, if a test pit is to be 30 ft deep, its dimensions at the top should be not less than 10 by 10 ft. Typical sheeting and bracing for an ordinary test pit are shown in Fig. 6.

On work of the first magnitude, test pits or test shafts are sometimes carried to great depths to confirm and elucidate the findings from the borings. In this country test pits are generally terminated when rock is reached or even before, but in some other countries it is customary to use test shafts in rock also.

20. Wash Borings. Wash boring and churn drilling are inexpensive methods of getting through the overburden, but they should not be depended on for showing the character of material passed through.

In wash boring there is a small pipe used on the inside of a larger pipe or casing. At the end of the inside pipe there is a nozzle sometimes with a chopping bit. Water under pressure is admitted to the inside pipe and it is jiggled up and down with the result that a mixture of water and material is discharged over the rim of the casing. This loosens the casing at the lower end, and the casing is then driven down a little farther, usually by a weight operated from a tripod, and so on, with ledge rock as the desired depth to be reached.

By catching the mixture of water and material as it comes out of the casing one may obtain samples of a portion of the material passed through which is both disturbed and washed. By carefully watching the wash water all the time one can sometimes get a fair idea of whether he is passing through sand and gravel, sand, or sand with silt and clay, etc. Altogether wash boring is usually the cheapest way to get through the overburden, it is the poorest way to learn anything very definite about the character of the overburden. In cases where drive sampling is being done, wash boring may be used for that portion of the hole where no samples are desired provided that proper precautions are taken to clean the washings out of the hole before the drive sampling is done. (See Arts. 23 to 35 of this chapter.)

21. Churn Drilling. In churn drilling, a barrel or spoon is utilized inside the outer casing. The spoon or barrel is of limited length and instead of the

method using a superabundance of water, as in wash boring, it uses only a relatively small amount of water in the casing. Sometimes the required water is furnished entirely by the natural water in the underground. The barrel or spoon has a flap valve or tail valve at or near its lower end, such that material may enter into the barrel, but cannot get out. A chopping bit of varied design is usually placed below the entrance to the barrel. The barrel and chopping bit are then jiggled up and down by means of a rope or cable attached to their upper end and the material is thus forced into the barrel. The casing is pounded down as the operation of the spoon continues. When the barrel is nearly full, it is removed and the material inside it dumped out. Most of the material passed through is present in the sample, but it is mixed up, partly segregated, washed, and just about as far from being in its natural condition as it possibly could be. A power-operated earth auger is also provided with some drilling rigs, and it is usually an inexpensive way of obtaining disturbed samples of the overburden.

The above is the usual well-drilling method. By the use of heavy equipment and special chopping bits, the method has been developed for penetrating even relatively hard ledge rock. Both wash-boring and churn-drilling methods permit taking drive samples at intervals.

An inexpensive method of subsurface investigation of overburden is furnished by the use of hand-operated earth augers ("Iwan" and other types). Samples are disturbed, but this may answer for most of the samples if one is investigating a borrow pit for instance. Depths of 20 to 40 ft may be reached in some soils by this method.

22. Rotary Drilling Through Overburden. Rotary core drilling is similar in principle to diamond drilling. Using some form of hardened-steel core bit with cutting edge, the driller may put down exploratory holes through some firm clays and compact sands and silts without casing the hole and may obtain a core of the material. Drilling fluid consisting of a rather thick mud formed from fine clay or bentonite is kept in the hole, and the rotating bit presses the drilling fluid into the walls of the hole, thus giving them sufficient strength so that they will stand up. The core is also smeared with the drilling fluid, but the impregnation is usually slight. The writer has seen compact sand cores obtained in this manner in which the drilling fluid impregnated less than $\frac{1}{16}$ in. into the sample. The method was developed in the oil fields but has also been used quite successfully in exploration work in materials such as those described above. If diamond drilling is to continue after rock is reached, the hole through the overburden is cased before the diamond drilling is started, as usually the walls cannot be relied on to stand up indefinitely. The use of the rotary drilling rigs for the exploration of overburden material has its limitations.

In very stiff clays the rotary drill is often useful. Excellent undisturbed samples of Trinity sand, a fine dense sand whose void ratio may be as low as 0.21, were obtained at the site of the Denison Dam, Texas and Oklahoma. The method cannot be successfully used for sampling coarse sands and gravels.

23. Undisturbed Sampling of Overburden. In order that the character, strength, and permeability of the overburden may be determined, it is necessary to obtain undisturbed samples of the material and put them through a series of tests in the laboratory. Such tests on undisturbed samples are particularly necessary for clays, silts, and very fine sands. It is usually of less importance to make tests of undisturbed samples of coarse sands and gravels because their stability or shear strength is usually adequate and permeability may usually be determined within the necessary range of accuracy by making tests on disturbed samples at approximately the same density as the material possessed before disturbance (see Chapter 16).

Test pits furnish one of the best means of obtaining samples which are for all practical purposes undisturbed. The only trouble is the expense of excavating at depth. At shallow depths, say 5 or 6 ft, the test-pit method is as cheap as any other. For the cost of one deep test pit, say 60 ft, however, one could take a large number of undisturbed or nearly undisturbed samples by drilling and driving methods.

To take an undisturbed sample of clay in a test pit, a small area in the bottom is carefully leveled off. Then the four sides of the sample, usually approximately 8 by 12 in., are carefully cut or carved and the surrounding material removed. The five sides of the sample are then paraffined and wrapped with two or three thicknesses of cheesecloth as the paraffin is applied. Then when all is ready the container is fitted over the top of the specimen, a sharp spade severs the specimen from its pedestal, and it is turned over, and its top, which is the bottom in its natural position, is paraffined and covered with fabric. The packing is then completed and the container is labeled. The specimen is then ready to go to the laboratory and may, if necessary, be stored for a considerable period of time before tests are made.

24. Drive Sampling of Clays and Silts. Because of the prohibitive cost of using test pits for obtaining all the undisturbed samples which it is frequently advisable to obtain, methods of taking undisturbed samples inside of casings have been developed.

The following descriptive data and the accompanying figures were furnished by Dr. M. Juul Hvorslev.[8] Dr. Hvorslev's work was done in connection with an extensive project sponsored by the Engineering Foundation for developing better methods and equipment for obtaining undisturbed samples of soils. For a fuller exposition of the subject, the reader is referred to the reports of the Committee on Sampling and Testing, American Society of Civil Engineers.

Other things being equal the greater the required diameter of the undisturbed sample, the greater the cost. As many of the desirable laboratory tests can utilize samples of small diameter, it is to be expected that for most investigations there will be a large number of borings utilizing, say a 2-in. sampler and a relatively small number utilizing, say a 4¾-in. sampler.

[8] Research Engineer, Committee on Sampling and Testing, Soil Mechanics and Foundations Division, American Society of Civil Engineers.

That the so-called undisturbed samples obtained and tested are sometimes in reality very much disturbed is indicated by Figs. 7 and 8. Fig. 9 shows a slide of a sample which is very nearly truly undisturbed.

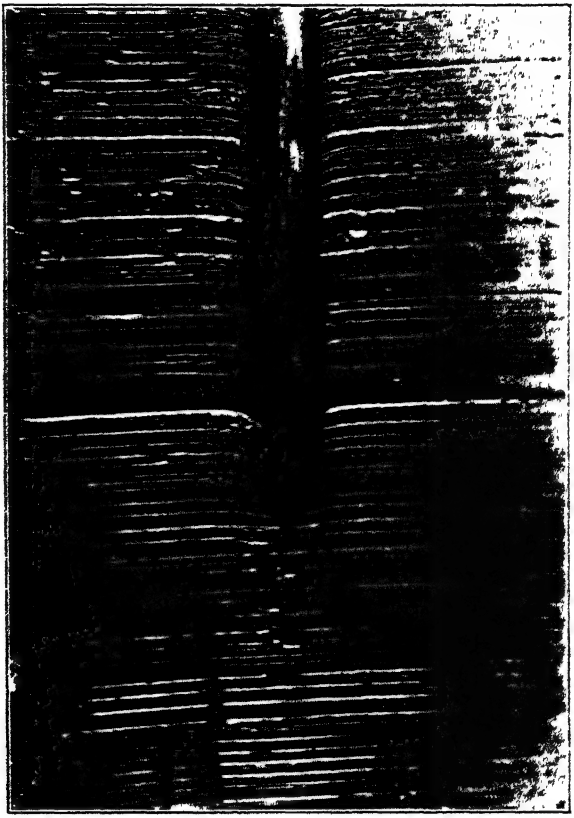


FIG. 7. Distortions due to ramming the sampling tube into the soil.
(Courtesy of Dr. M. Juul Hvorslev.)

25. Samplers for Silt and Clay. On account of space restrictions, only one small-diameter sampler and one large-diameter sampler (4¾-in.) for final undisturbed sampling, both to be used in cased boreholes, will be described here. These samplers and the appurtenant sampling equipment are a part of the recent improvements in sampling equipment and methods developed under a project sponsored by the Engineering Foundation. The equipment shown in Figs. 10 to 13 was developed by Dr. M. Juul Hvorslev, in cooperation with H. A. Mohr, District Manager of the Raymond Concrete Pile Company, the Providence District of the U. S. Engineer Department, and the U. S. Waterways Experiment Station in Vicksburg, Miss.

26. Prevention of Failure of Soil During Sampling. The first requirement in undisturbed sampling is that failure or a change in the properties of the soil adjacent to the bottom of the borehole must be prevented. When sampling above the ground-water level, the borehole should be kept dry; otherwise the water content of partly saturated soils will be changed and their apparent cohesion destroyed, which may result in loss of the sample during the withdrawal

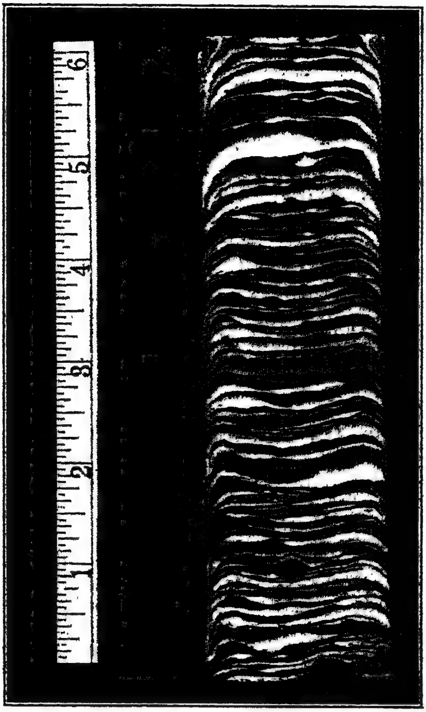


FIG. 8. A disturbed "undisturbed" sample of varved clay obtained by ramming or intermittent driving. (*Courtesy of Dr. M. Juul Hvorslev.*)

of the sampler. As soon as the sampler is withdrawn the casing must be driven to the new bottom of the borehole. Boreholes in firm clays are often kept dry but when softer soils or soils with but a small amount of cohesion are encountered, the borehole must be filled with water.

In general, it is safest to fill the borehole with water as soon as the groundwater level is reached. An actual "blowing in" of the borehole with the soil rising many feet in the hole is a very serious occurrence, since the soil may thereby be disturbed to an unknown depth below the bottom of the hole. When such soils are encountered, collapse of the borehole may be prevented by lowering the casing to the elevation of the sampler cutting edge and simultaneous removal of the soil between the casing and the sampler by means of jetting or ring-shaped augers before the sampler is withdrawn.

Improper cleaning of the borehole before the actual sampling operation is frequently the cause of a very serious disturbance of the upper part of the recovered sample. Boreholes filled with water are commonly cleaned by means of bailers, sand pumps, or jet pipes. A strong water jet directed toward the bottom of the borehole may erode the soil to a considerable depth, and the funnel-shaped hole thus created will often be filled with mixed and coarse

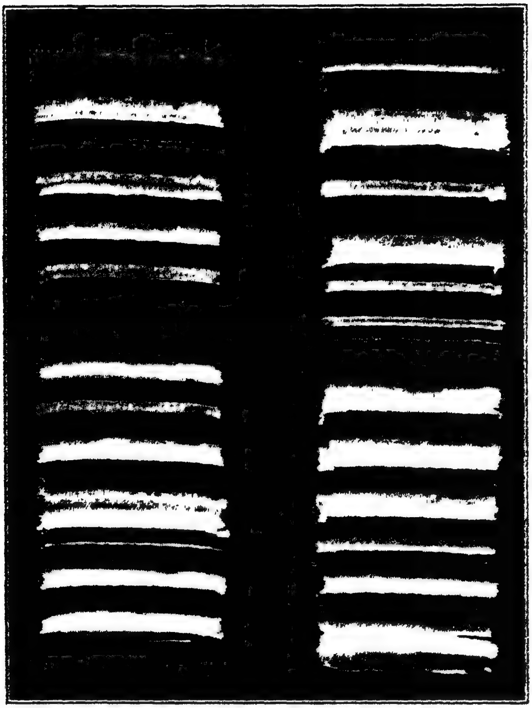


FIG. 9. A practically undisturbed sample of varved clay obtained by jacking (continuous steady motion of sampler into soil). (*Courtesy of Dr. M. Juul Hvorslev.*)

material when the jet pipe is withdrawn. The jet pipe should, therefore, be so constructed that the jet is directed toward the sides of the casing and it should not be operated below the bottom of the casing. In general, the above-mentioned equipment will not remove clay adhering to the inside of the casing, and coarse material still in suspension will settle while the cleaning tools are being withdrawn and the sampler lowered.

27. Clean-Out Auger and Calyx. To overcome the above difficulties the clean-out auger for 6-in. casing, shown in Fig. 10, was developed. A similar auger was also developed for 2½-in. casing. The clean-out auger is built around a jet pipe with jet holes in the wall of the cap at its lower end. Fastened to the jet pipe by means of fins at the top and a plug at the bottom is a larger thin-walled tube or calyx with two sharpened fins or scrapers on the outside.

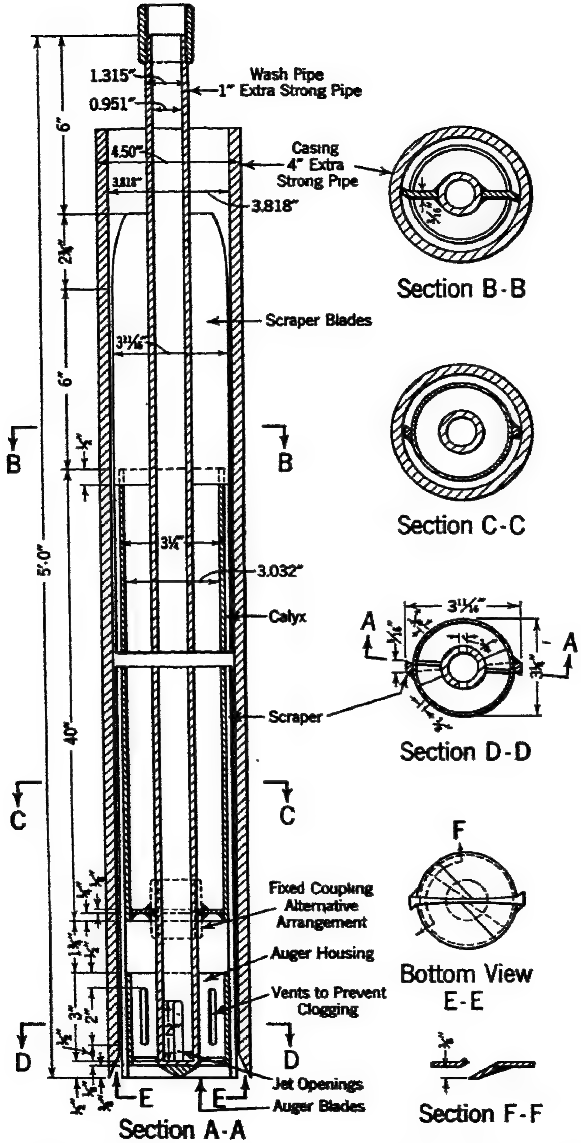


FIG. 10. Jet auger and scraper with calyx. For cleaning of boreholes just previous to sampling. For use where 6 in. casing has been used in holes. (*Hvorslev.*)

A short heavier section of tubing with large vents for water and soil is welded to the bottom of the calyx. Screwed to this tubing is a shoe with the bottom formed as a flat, double-spiral auger. In the walls of the shoe are several narrow slots to allow circulation of water and prevent grouting or wedging of material between the casing and the lower part of the shoe.

When the jet pipe is rotated, material loosened by the side scrapers or forced into the shoe by the action of the auger blades will be carried away by the wash water flowing upward at high velocity between the calyx and the casing. When the soil-laden water passes the top of the calyx, the velocity is suddenly reduced to a fraction of its former value, and the coarse material and clay lumps will be deposited in the calyx while the fine material will be carried upward and out of the casing by the wash water.

In order to retain gravel which has entered the shoe and has not been removed by the wash water, the bottom of the shoe may be covered with a rubber pad, which is slit and fastened in such a manner that it will allow material to enter but prevents it from dropping out through the openings between the auger blades.

28. Small-Diameter Sampler (2-In.). The sampler shown in Fig. 11 is a further development of the thin-walled seamless steel tubing samplers originated by H. A. Mohr. The lower end of the tubing is drawn in and reamed so that a sharp cutting edge is formed with a diameter which, according to the soil conditions, is from ½ to 2 per cent smaller than the inside diameter of the tubing. The tubing is fastened to the drill rod by means of two Allen set-screws, and a tight seal is formed by the leather cup at the lower end of the adapter. Unhindered flow of water from the tubing is provided by the three streamlined vents and by the reduction of the outside diameter of the upper end of the adapter. The increase in hydrostatic pressure over the sample during rapid driving is thereby reduced to a negligible amount. During the withdrawal of the sampler a vacuum is produced over the sample by means of the piston-type check valve which replaces the commonly used but unreliable ball check valve. When the sampler is lowered into a water-filled borehole, the hydrostatic pressure will force the piston into the top of the adapter, where it will not interfere with the flow of water through the vents.

When the sampler has been driven, the top of the drill rod is connected to a pump and hydrostatic pressure is applied over the piston, which is forced slowly down into its seat. The clearance between the piston and the adapter is so adjusted that water can leak past the piston in an upward direction but on account of the leather cup in the piston not in a downward direction. Just before and during the first part of the withdrawal the sampler should be rotated in order to separate the sample from the subsoil.

After withdrawal the set screws are removed and the vacuum over the sample is thereby broken. The tube and adapter can then be separated. Soft and disturbed material in the top of the tube is removed, whereupon the tube is cut off above the sample and sealed with paraffin and caps or metal disks and adhesive tape. In the laboratory the tube is cut with a fine-toothed hacksaw

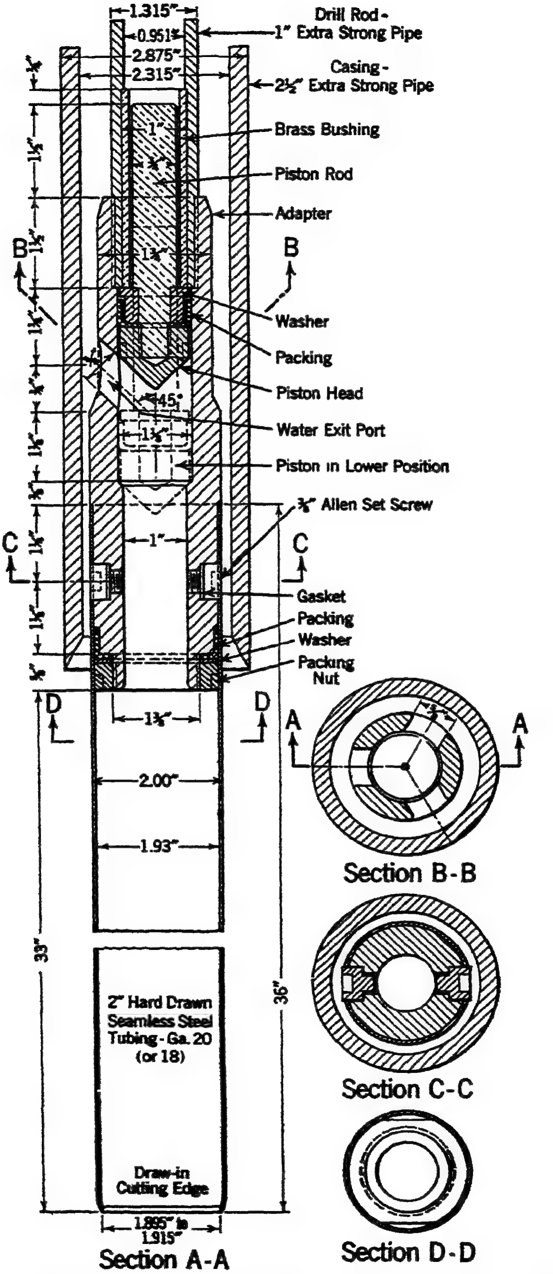


FIG. 11. Two-inch tubing sampler with piston check valve. (*Hvorslev.*)

into sections about 6 in. long; the burrs at the ends are removed with a scraper and the sample pushed out by means of a plunger with a close-fitting cork or wood head.

29. Thin-Walled Steel Tubing for Large-Diameter Samples. Thin-walled steel tubing has also been used successfully for obtaining large undisturbed samples up to 8 in. in diameter, but steel tubing is not desirable unless testing is to be done promptly, as corrosion may affect the properties of the material and interfere with the removal of the sample. For small-diameter samples this is not so serious because the simpler tests for which they are taken can usually be made promptly. It is preferable to preserve large-diameter samples, which frequently cannot be tested promptly, in tubes of noncorrosive materials. The steel tubing may be replaced with brass tubing, but a greater wall thickness will then be required. It is also possible to transfer the sample from the steel tubing to containers of other material, but the additional manipulation of the sample may cause disturbance of the soil. The common alternative is a composite sampler in which the steel tubing has a thin-walled liner of noncorrosive materials.

In certain soils it is also difficult to separate a large sample from the subsoil by rotating the sampler, and it may be necessary to provide a sampler shoe with a cutting wire. The liner and special shoe will increase the effective wall thickness of the sampler and thereby the tendency to disturb the soil. This disadvantage can be avoided by using a Swedish piston-type sampler. This type of sampler has many other advantages which more than compensate for the additional time required to assemble and disconnect the piston rod each time a sample is taken.

30. Large-Size Piston-Type Sampler. A 4¾-in. piston-type sampler with a brass liner is shown in Fig. 12. The barrel is connected to a sampler head which has three water exit ports and a split cone wedge for clamping the piston rod. The lower end of the barrel has a slight outside upset to strengthen the joint with the removable cutting edge and to provide an outside clearance which will reduce the friction between the barrel and the surrounding soil. The cutting edge is removable and its diameter is from 0.5 to 2 per cent smaller than the inside diameter of the liner. It is advisable to have several cutting edges with varying diameters on hand so that the inside clearance—the difference between the diameters of the liner and the cutting edge expressed in per cent of the former—can be varied according to the soil conditions. If the inside clearance is too small, the sample will be disturbed, and if it is too large, the sample may be lost during the withdrawal of the sampler. Grooves for a single-loop cutting wire are provided above the cutting edge and on the outside of the barrel. The groove above the cutting edge is closed by the liner during the lowering of the sampler, but there is a slight clearance between the top of the liner and the sampler head, and soil entering the sampler will move the liner upward and free the cutting-wire groove.

The liner consists of two parts; the lower 4-ft section serves as a container for the sample proper, while the upper and permanent section accommodates

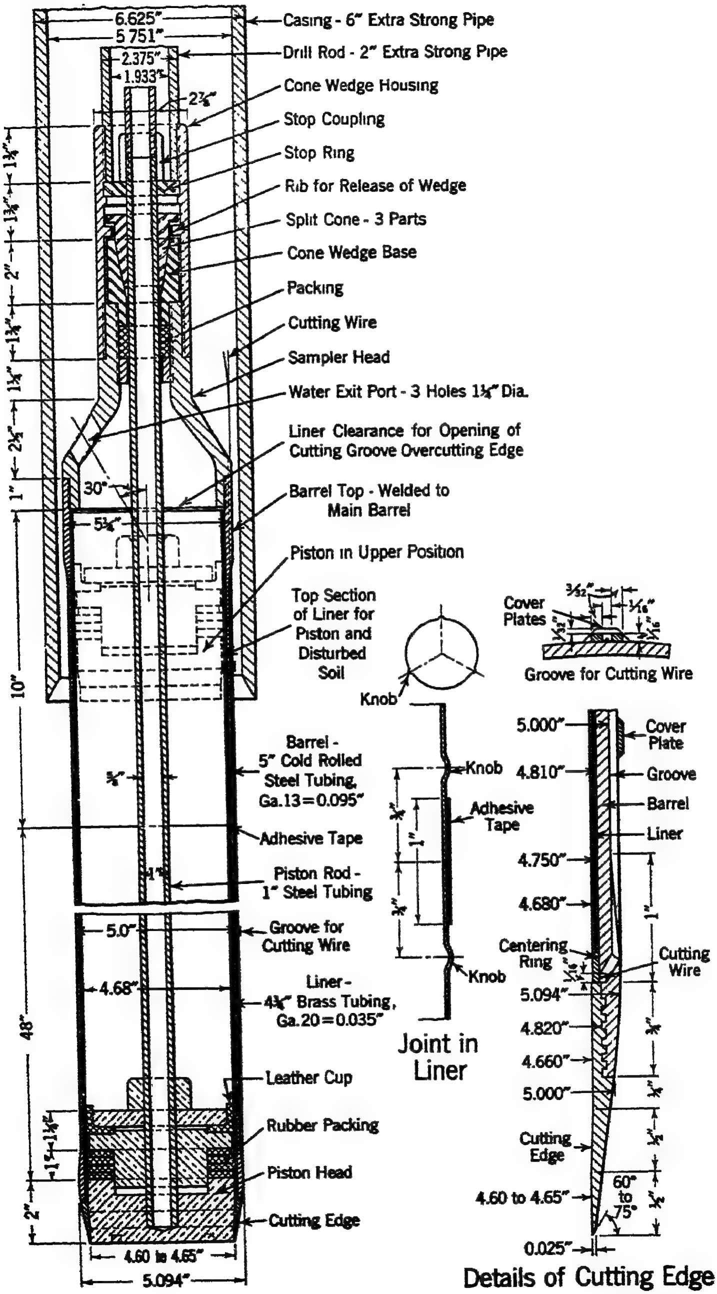


FIG. 12. Four and three-fourths in. piston type sampler. (*Hvorslev.*)

the piston in its upper position and serves as a reservoir for disturbed soil. The joint between the two sections is sealed with adhesive tape. Ample clearance is provided between the barrel and the liner to facilitate removal of the latter. The liner is properly centered by means of centering knobs at its upper end and a centering ring at its lower end. This ring will also prevent soil from entering the space between the liner and the barrel.

In some soils it may not be possible to obtain a 4-ft undisturbed sample in a single drive; the 10-in. upper section of the liner can then be replaced with a longer section and the lower section shortened a corresponding amount. For reasons of strength it would be preferable to use a single section liner 58 in. long; the upper part of the liner must then be wasted and it will be necessary to cut the liner below the piston before it is removed in order to break the vacuum between the piston and the sample.

The piston is flush with the cutting edge during the lowering of the sampler; it is held in this position by water entering through the vents and by a coupling on the piston rod and a stop ring in the sampler head. The piston will prevent clay shavings and other extraneous matter from entering the sampling tube. When the sampler reaches the bottom of the borehole, the piston rod is clamped to the upper edge of the casing and the piston will thereby be held at a constant elevation while the sampler is forced around it into the soil.

During the first part of the drive the piston will prevent entrance of excess soil displaced by the cutting edge and the barrel; during the last part of the drive the pressure between the piston and the top of the sample will decrease until a vacuum is established. This will result in the friction between the sample and the liner being less. The load on the soil below the sampler and the disturbance of the sample will also be decreased. It will thus be possible to obtain a longer undisturbed sample in a single drive. A downward movement of the piston during the withdrawal of the sampler is prevented by the cone wedge in the sampler head; the uncoupling of the piston rod is thereby facilitated and the vacuum over the sample maintained. After the sampler is withdrawn the cutting edge is removed and the cone wedge housing partly unscrewed; the grip of the wedge is thereby released, and the liner can then be pushed out of the barrel by means of the piston rod.

31. Importance of Fast Continuous Pushing of Sampler. The investigations for the Committee on Sampling and Testing [9] have clearly demonstrated that a minimum of disturbance and a maximum length of sample in a single drive is obtained when the sampler is pushed into the soil in a fast uninterrupted motion. Fast continuous pushing can be accomplished by means of a hydraulic jack having a stroke at least equal to the effective length of the sampler or by means of the block and tackle arrangements shown in Fig. 13. The figure also shows an arrangement for clamping the piston rod to the casing during the drive.

32. Application of Samplers Described. The wall thickness of the samplers described is small in order to reduce the disturbance of the soil. The samplers

[9] Soil Mechanics and Foundations Division of American Society of Civil Engineers, in connection with a project sponsored by the Engineering Foundation.

the piston in its upper position and serves as a reservoir for disturbed soil. The joint between the two sections is sealed with adhesive tape. Ample clearance is provided between the barrel and the liner to facilitate removal of the latter. The liner is properly centered by means of centering knobs at its upper end and a centering ring at its lower end. This ring will also prevent soil from entering the space between the liner and the barrel.

In some soils it may not be possible to obtain a 4-ft undisturbed sample in a single drive; the 10-in. upper section of the liner can then be replaced with a longer section and the lower section shortened a corresponding amount. For reasons of strength it would be preferable to use a single section liner 58 in. long; the upper part of the liner must then be wasted and it will be necessary to cut the liner below the piston before it is removed in order to break the vacuum between the piston and the sample.

The piston is flush with the cutting edge during the lowering of the sampler: it is held in this position by water entering through the vents and by a coupling on the piston rod and a stop ring in the sampler head. The piston will prevent clay shavings and other extraneous matter from entering the sampling tube. When the sampler reaches the bottom of the borehole, the piston rod is clamped to the upper edge of the casing and the piston will thereby be held at a constant elevation while the sampler is forced around it into the soil.

During the first part of the drive the piston will prevent entrance of excess soil displaced by the cutting edge and the barrel; during the last part of the drive the pressure between the piston and the top of the sample will decrease until a vacuum is established. This will result in the friction between the sample and the liner being less. The load on the soil below the sampler and the disturbance of the sample will also be decreased. It will thus be possible to obtain a longer undisturbed sample in a single drive. A downward movement of the piston during the withdrawal of the sampler is prevented by the cone wedge in the sampler head; the uncoupling of the piston rod is thereby facilitated and the vacuum over the sample maintained. After the sampler is withdrawn the cutting edge is removed and the cone wedge housing partly unscrewed; the grip of the wedge is thereby released, and the liner can then be pushed out of the barrel by means of the piston rod.

31. Importance of Fast Continuous Pushing of Sampler. The investigations for the Committee on Sampling and Testing [9] have clearly demonstrated that a minimum of disturbance and a maximum length of sample in a single drive is obtained when the sampler is pushed into the soil in a fast uninterrupted motion. Fast continuous pushing can be accomplished by means of a hydraulic jack having a stroke at least equal to the effective length of the sampler or by means of the block and tackle arrangements shown in Fig. 13. The figure also shows an arrangement for clamping the piston rod to the casing during the drive.

32. Application of Samplers Described. The wall thickness of the samplers described is small in order to reduce the disturbance of the soil. The samplers

[9] Soil Mechanics and Foundations Division of American Society of Civil Engineers, in connection with a project sponsored by the Engineering Foundation.

are primarily intended for use in silt and clay and are not strong enough for sampling of very hard and partly cemented soils or soils containing an appre-

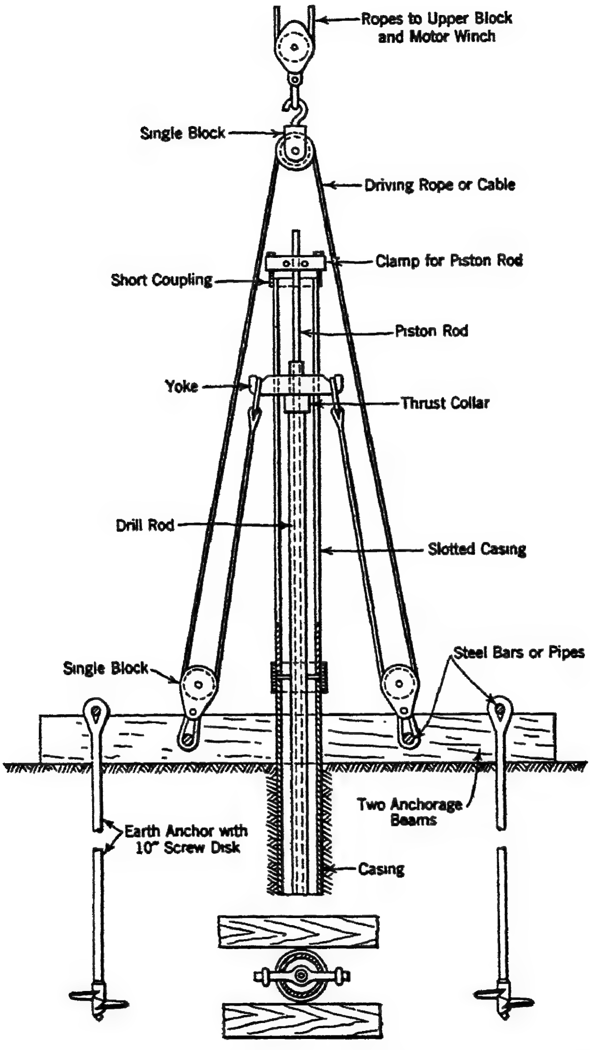


FIG. 13. Soil sampling by pushing with block and tackle. (*Hvorslev.*)

ciable amount of coarse gravel. In spite of the vacuum over the sample prc vided by means of the pistons, the samplers will not always retain very sof soils or saturated, entirely cohesionless soils. After the sampler has bee driven it is advisable to wait 10 to 15 min before withdrawing it in order t allow full friction to be established between the sample and the sampler. Th

sampling tube should be raised very slowly through a water-filled borehole in order to avoid erosion by turbulent water and a material decrease in the hydrostatic pressure below the sampler. The longer the sample, the smaller the danger of losing part of it or all of it during the withdrawal. In difficult soils it will also be helpful to use some suitable method of conveying compressed air to the underside of the sample or lowering the casing and simultaneously removing the soil between the casing and the sampler before the latter is withdrawn. When these methods do not suffice to prevent loss of the sample, it is necessary to use samplers with core retainers or, in case of cohesionless soils, the Fahlquist method of freezing a short section near the bottom of the sample before the sampler is withdrawn.

33. Sampling of Cohesionless Materials. Some fine sands, if sufficiently loose, are subject to flow under load upon slight disturbance or may settle unduly on application of load. Consequently it is desirable to perform tests on samples in their natural condition in order to determine void ratio and density, critical void ratio and shear strength. In the past, however, great difficulty has been experienced in procuring samples of cohesionless materials in anything like their natural condition in the ground, particularly where such materials are below ground-water level.

When fine, loose, cohesionless materials are situated below the ground-water level, procuring even reasonably undisturbed samples becomes extremely difficult, expensive, and uncertain, especially when these materials are very fine and are more or less uniform in size. Unfortunately, materials such as these, if they occur in a foundation, present difficult problems to the engineer.

Various sampling tubes for obtaining "undisturbed" samples of such materials have been devised. Spring fingers and other devices are sometimes used to retain the sample in the tube. At other times the suction created by a piston is utilized, and at still others valves are used. With most schemes one cannot be sure that the sample contains all the material intercepted between two definite elevations or that all the sample is retained in a practically undisturbed condition.

In connection with the investigation of the slide in the Fort Peck Dam, the cohesionless material was first frozen by methods similar to those used in sinking shafts. Then 30- and 36-in. holes were bored in the frozen cohesionless material, using a Calyx drill, and the large-sized undisturbed cores were obtained and studied. The stratifications of the undisturbed samples thus obtained provided valuable information. Although this method of obtaining undisturbed samples of cohesionless material is expensive there are conditions under which its use is justified in order to obtain truly reliable data for design purposes.[10]

The usual methods of sampling of cohesionless fine material result in so much disturbance that one cannot determine the natural over-all void ratio or density of a sample with any assurance. The sampling method described

[10] See Report on "The Slide of a Portion of the Upstream Face of The Fort Peck Dam," July 1939, Corps of Engineers, U. S. Army.

below overcomes these objections in a satisfactory manner. The plunger which remains stationary while the sampling tube is being jacked down, definitely fixes the elevation of the top of the sample in the ground, and the frozei soil plug fixes the bottom elevation so that the engineer knows he has all of th material between these elevations.

34. Fahlquist Method for Sampling Cohesionless Materials. The bes device for taking "undisturbed" samples of fine cohesionless materials (sand: and silts) which has come within the observation of the authors is that devel oped by Frank E. Fahlquist, Senior Geologist, Providence District, Unitec States Engineer Department, and his Associates, with advice as to particula features from Dr. M. Juul Hvorslev.

Figs. 14 and 15 indicate the procedure by which samples are obtained and th essential features of the equipment.

The scheme provides for (1) elimination of all impact at the site, such a driving of sampler and casing, or other disturbing vibrations, (2) obtainin soil sample within a thin-walled piston-type sampler, penetration of th sampler being accomplished by jacking, and (3) forming a temporarily im movable frozen soil plug in the bottom of the sampler, which will preven movement and loss of the sample. The equipment is comparatively simple i construction, and the method of operation is not involved, requiring only care ful and attentive workmanship.

In this scheme the sampler is advanced in a steady fast jacking action b means of a block and tackle attached to a tripod over the borehole. Reactio pulleys anchored into the ground are utilized so that when a pull is exerted o the fall line, a steady downward pressure is exerted on the sampler, as illus trated in Fig. 13.

35. Procedure in Use of Fahlquist Sampler.

(*a*) The 10-in. outside casing is jetted and turned down by hand throug the overburden which it is not desired to sample. A 6-in. flush jointed casin is then placed and centered within the 10-in. casing. Jet pipes are inserted i the annular space between the 6-in. and 10-in. casings. These jet pipes, whic are utilized in sinking the 6-in. casing, should always be kept 10 to 15 ft abov the cutting edge of the 6-in. casing. The 6-in. casing is washed and turned c screwed down only to the point where it is desired to start sampling, all a indicated in operation 1, Fig. 14.

(*b*) Within the 6-in. casing there is inserted a clean-out auger having cuttin teeth on the bottom and provided with jets discharging in a horizontal direc tion. The clean-out auger rod is simply a pipe through which water is force for the horizontal jets of the auger and which is used to rotate the auger b hand. Immediately above the auger and attached to the auger rod is a caly or shroud provided to catch suspended particles while cleaning out within tl 6-in. casing. This prevents accumulation of sand and fines at the bottom the hole, which otherwise would be deposited on top of the strata to be san pled. By rotating the auger and operating the jet, the material inside of tl 6-in. casing is removed and transported upward out of the casing by tl

ascending wash water. The cleaning-out process is continued only until the bottom of the 6-in. casing is reached. The clean-out auger is then removed (operation 2, Fig. 14).

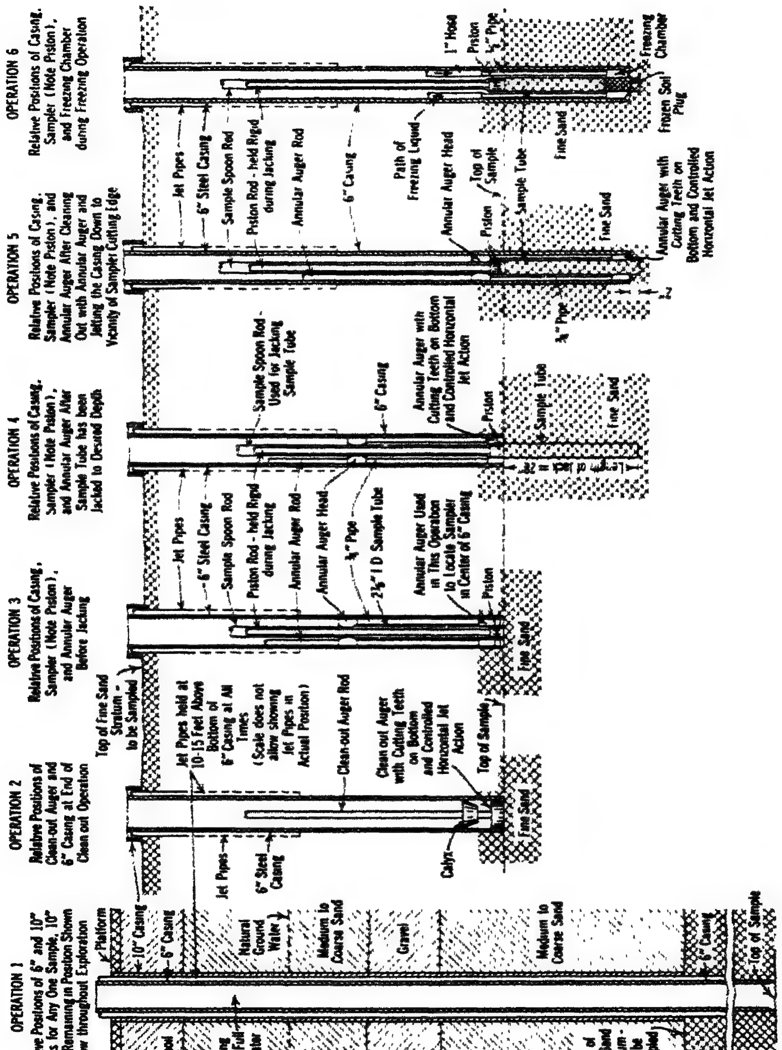


FIG. 14. Procedure with Fahlquist sampler in cohesionless materials. (*Courtesy of U. S. Engineer Office, Providence, R. I.*)

(*c*) The $2\frac{7}{8}$-in. inside-diameter sampling tube, in which is placed the piston with its base flush with the bottom of the sampler, is then assembled with the annular auger on the outside of the sampler. The assembly, which fits neatly

inside of the 6-in. casing, is then lowered to the bottom of the hole as indicated in operation 3, Fig. 14.

(*d*) The 36-in. sampler with plunger remaining stationary is then forced down into the stratum to be sampled by a steady fast jacking action, as already described, for a distance of 28 in. below the bottom of the 6-in. casing, as indicated in operation 4, Fig. 14.

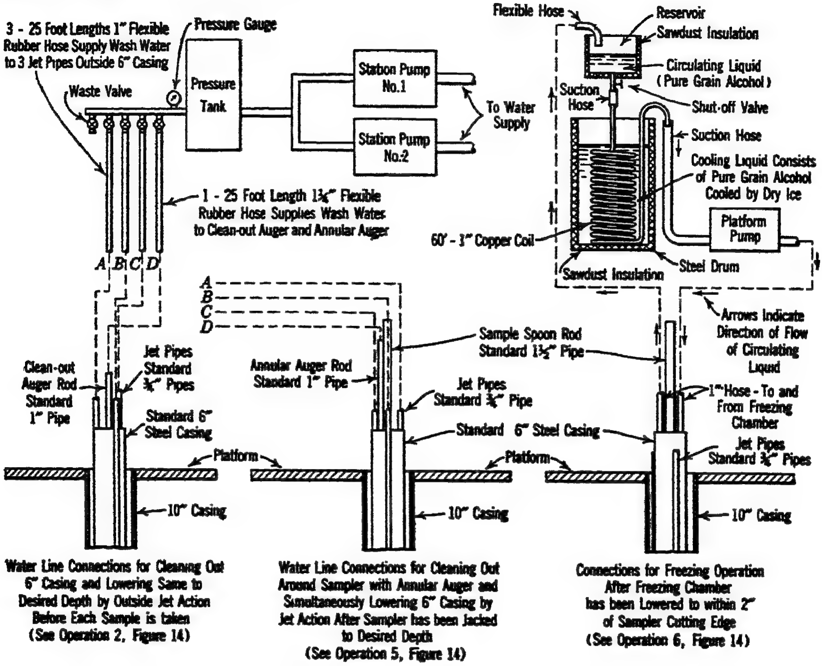


FIG. 15. Fahlquist auxiliary surface equipment. For sampling cohesionless soils. (*Courtesy of U. S. Engineer Office, Providence, R. I.*)

(*e*) The annular auger with its horizontal jets is then operated to clean out material in the annular space between the sampler and the 6-in. casing, and allow for simultaneous lowering of the casing. The operation is stopped when the 6-in. casing reaches a point about 2 in. above the cutting edge of the sampler, as indicated in operation 5, Fig. 14.

(*f*) The annular auger is then withdrawn from the space (which necessarily extends all the way to the surface) between sampling tube and the 6-in. casing. A freezing chamber is then lowered to a position around the sampler and at the bottom of it. By methods clearly indicated in Fig. 15 alcohol at a temperature of about −30° C is circulated through the freezing chamber for about 15 min, which results in a plug about 7 in. long being frozen in the end of the sampler, as shown in operation 6, Fig. 14.

If the freezing chamber should be frozen to the 6-in. casing, it is readily freed by a shot of hot water. The sampler containing the soil sample now confined

by the frozen plug is next removed from the borehole and laid on a bench. Here the freezing chamber is removed, and the vacuum between the top of sample and bottom of piston is relieved by removing a threaded plug from the piston. The piston and head are then removed from the sampler and the tube is thoroughly sealed with paraffin and taken to the laboratory. If a continuous record is desired another sampling tube is then utilized and operations 3 to 6 are repeated until the desired depth is reached.

Determination of the void ratio and per cent saturation are made in the laboratory by removal of soil within the tube in increments of about 1¼ in. The number of determinations made per sample ranges from 15 to 20, depending upon the length of the small samples removed and the size of the frozen soil plug. While still in a frozen condition, the plug may be sliced, photographed, paraffined, and preserved for future reference.

The equipment and method described above have been used successfully in obtaining undisturbed samples down to a maximum depth of 50 ft below the ground surface, or 45 ft below the water table. Such a boring, with sampling intervals of 5 ft and a drilling crew of five men, including an inspector, have been completed within 6 days.

36. Diamond Drilling. For drilling exploratory holes in ledge rock the diamond core drill is almost invariably the most suitable tool. A core of the strata passed through is obtained which may be studied by the geologist and the engineer so that a true picture of the foundation, including the presence of faults, seams, cavities, and changes in the quality of the rock, may be obtained.

37. The Diamond Drill. The diamond drill is rotated by means of a gasoline engine, steam engine, Diesel engine, electric motor, or air motor. At the head of the drilling machine is a screw feed or hydraulic feed for maintaining just the desired amount of pressure on the drill rods. There is also a hoist for raising and lowering the drill rods, and a tripod is usually set up over the hole to facilitate this. The drill rods are of special heavy construction with flush joints.

The cutting edge is a steel ring set with black diamonds commonly called "carbon," or with an impure crystalline form of diamond known as "Bortz." Above the ring, or diamond bit as it is commonly called, is a core barrel into which the core passes as the bit cuts down into the rock. At the bottom of the core barrel is a split ring spring core lifter, so that when the column of drill rods, core barrel, and bit are lifted out, the core will be caught and retained. When the assembly is in action, water is pumped down on the inside of the drill rods to keep the diamond bit cool and to carry the rock cuttings up on the outside of the drill rods. When a double core barrel is used, the inner tube of the core barrel fits down over the core and its upper end is plugged so that water from the inside of the drill rods passes down between the inner and outer tubes of the core barrel and does not touch the core at all except at the cutting edge of the bit. This is because it has been found that the mere presence of water under high pressure tends to destroy the cores of the softer rock. It is good practice to use the double tube core barrel in exploratory work in all

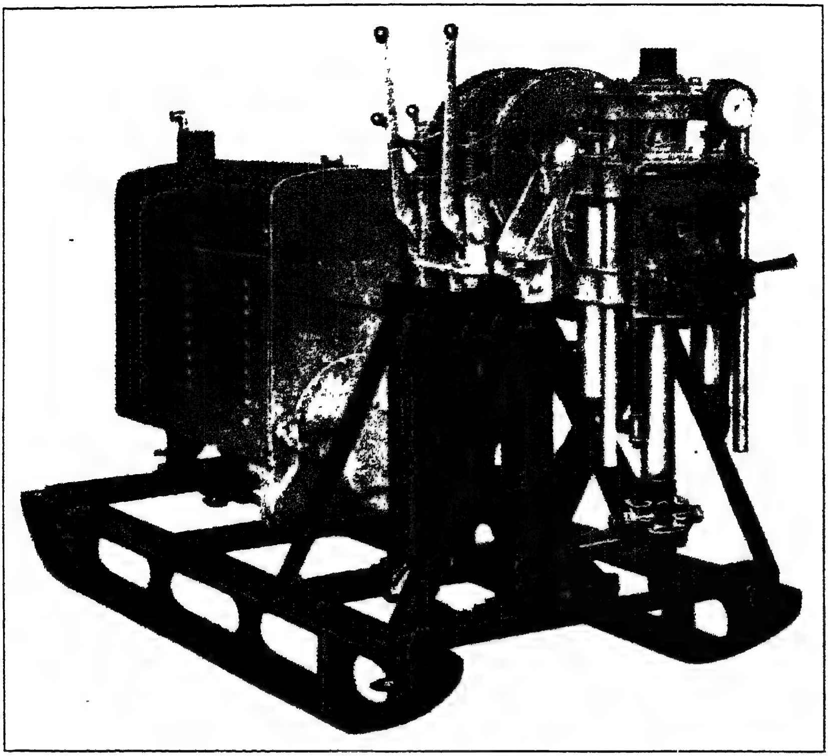


FIG. 16. Diamond-drill rig with double hydraulic swivel head. (*Sprague & Henwood.*)

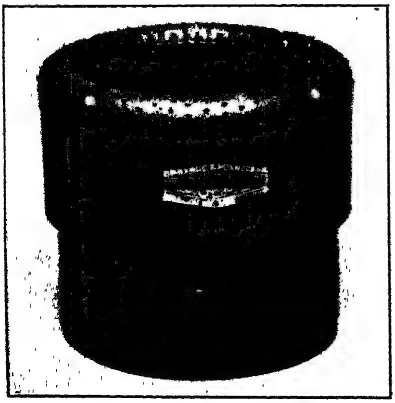


FIG. 17. Cast-set (Bortz) diamond bit. (*Sprague & Henwood.*)

except the very hardest and most massive rocks as higher percentages of recovery are thereby promoted.

Fig. 16 shows a modern diamond-drilling rig, Fig. 17 a cast-set diamond bit, and Figs. 18 and 19 the assembly of diamond bit core lifter and core barrel for both a single-tube and a double-tube core barrel.

38. Improved Efficiency of Diamond Bits. Recent years have shown considerable improvement in the efficiency of the drilling machines and also in the tools and the diamond bit itself. Until recently each field job required one or more diamond setters to keep the bits properly set. As diamond setting was a

very highly skilled trade, this expense was considerable. The bits now used generally carry very much smaller diamonds and when the bit wears down, it is generally sent back to the factory and another one is substituted.

Approximately 90 per cent of all diamond drilling at the present time is done with a "Bortz diamond bit." Nearly all of these bits, especially in the standard sizes, are mechanically set or "cast-set" instead of being of the old hand-set type. The diamonds are cast in a matrix of an alloy which can be hardened by a heat-treating process after casting to make it more resistant to the abrasive action encountered during drilling operations. There are several different types of cast-set Bortz diamond bits on the market at the present time.

39. Size of Core and Holes. The mistake is frequently made in exploratory drilling of using too small a hole with a resulting small-diameter core. In general the larger the core the greater the assurance of a high percentage of recovery. It should be remembered that what is desired is the maximum amount of information at an economical cost. The pursuit of the maximum amount of footage at minimum cost often defeats the entire purpose of exploratory drilling. In general the double core barrel should be used and the diamond bit should not be smaller than the standard "BX" bit, which gives a core approximately 1⅝ in. in diameter.

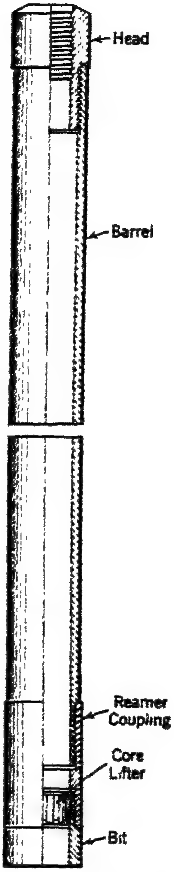


FIG. 18. Diamond-drill single-tube core barrel.

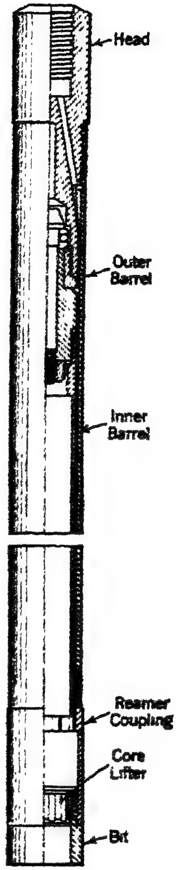


FIG. 19. Diamond-drill double-tube core barrel.

(*Courtesy of Sprague & Henwood.*)

The following sizes are in accord with the adopted standards of the Diamond Core Drill Manufacturers Association.

NATIONAL BUREAU OF STANDARDS, COMMERCIAL STANDARD CS 17–42

Size Designation Core Barrel Bit	Diamond Bit Outside Diameter (inches)	Approximate Diameter of Hole (inches)	Approximate Diameter of Core (inches)
EXT	$1\frac{27}{64}$	$1\frac{1}{2}$	$\frac{15}{16}$
AXT	$1\frac{53}{64}$	$1\frac{7}{8}$	$1\frac{5}{16}$
BX	$2\frac{19}{64}$	$2\frac{3}{8}$	$1\frac{5}{8}$
NX	$2\frac{59}{64}$	3	$2\frac{1}{8}$

40. Importance of Accurate Boring Data. A high percentage of recovery is greatly to be desired as it decreases the amount of guessing which is necessary. If the total length of core recovered equals the depth of the hole in the rock, the recovery is said to be 100 per cent. In many rocks, by the use of proper methods and equipment with proper diameter of bit, 100 per cent recovery is not only possible but is frequently approached. In actual practice 80 to 90 per cent recovery is quite common.

The importance of accurate boring data is indicated by the following recent incident. Some exploratory work for a tunnel showed the strata to consist of hard quartzite interbedded with a soft greasy shale. In the diamond drilling an "EX" bit (about the smallest commercial size) was used, giving a core of about ⅞ in. in diameter. A single core barrel was used. As a result the soft greasy shale was almost entirely ground up and washed away so that only the quartzite was recovered. The cores were placed in boxes without spacers to indicate the missing strata. The actual recovery was approximately 65 per cent, but practically all of the quartzite was recovered.

The contract drawings followed the apparent indications of the core boxes and showed massive and continuous quartzite. The core boxes and the drawings mentioned constituted the information available to bidders.

The information was grossly misleading, and the successful bidder eventually recovered a substantial sum representing the additional cost of driving a tunnel through the difficult formation encountered over what it would have been with sound firm rock which the information available led him to expect.

Many incidents similar to the above, where misleading data relative to exploration work have resulted in costly extras, might be cited. They emphasize the importance of thorough exploratory work, properly interpreted. However, the main reason for sparing no pains to obtain as true a picture of subsurface conditions as practicable is that only in that way can the engineer design his structures with assurance of both safety and economy.

41. Large Drill Holes. In the exploration of a dam site it has become rather common practice to put down one or more 30- to 36-in. diameter drill holes generally drilled by shot drilling methods. These holes have the very great advantage that they leave no doubt at all about the character of the rock passed through. The engineer or geologist can have himself lowered to the bottom of the hole in a boatswain's chair or in a cage operated from a hoist and can study every inch of the walls. A true picture of all the strata passed through is thus obtained. The only thing against these holes is their cost, which is usually $40 to $70 per ft or 15 to 40 times as much as ordinary diamond-drill holes. As an alternative to a shaft in ledge rock they are a satisfactory and economical substitute.

42. Care of Cores and Samples. The cores and samples should be taken, handled, and filed with the utmost care. An inspector versed in engineering geology should be constantly in attendance on the drilling operations, and he should allow no detail of the drilling to escape him. Substantial core boxes of uniform size should be constructed. Four ft long by about 12 in. wide is a

usual and convenient size. The box should be divided into troughs just wide enough to take the core. As soon as a core is brought up it should be filed. Small stickers on which the inspector writes his elevations, classifications, and other data should immediately be fastened on the partitions in the box, so that the box forms a complete visual record of the hole. As soon as the box is filled, the cover should be screwed on, the box properly labeled with the designation of the hole and the limiting elevations of the core it contains, and held pending the final examination of the core and samples by the engineer and the geologist. After that it should be permanently put in storage and not destroyed. The safety of foundations is sometimes questioned by the public authorities even after dams have been in service many years, and in such an event the owners will find it of the utmost value to have this evidence at hand.

43. Keeping of Records. The inspector assigned to the drilling should have some knowledge of geology, and the matter should receive the close supervision of the engineer. The record should be kept in complete detail, and nothing of significance should be allowed to escape. For instance, the loss of the water used in the operation of diamond drilling is sometimes a serious matter and might indicate the existence of a large open seam. Holes should be numbered consecutively and be designated by their position in the system of coordinates, thus: Hole 22 $\frac{\text{N22452.3}}{\text{E14451.6}}$. It is often convenient to add a letter to the number to designate the nature of the hole; thus D for diamond-drill hole, P for test pit, and W for well-drill hole. In addition to a record which forms a log of the hole, the inspector should turn in a daily report, giving the holes being drilled, progress for the day and data concerning conditions and incidents of moment.

44. Pressure Testing Device. After drilling has been completed and before the pump and machine are removed the hole should be tested out under water pressure to determine the presence and location of any open seams and to determine how nearly tight the rock is.

A pressure testing device which has been used by the author on many investigations is shown in Fig. 20. The device consists of a smaller pipe placed within a larger. In Fig. 20 the dimensions given apply to a hole approximately 2 in. in diameter. If the hole is larger than 2 in. it is merely necessary to increase the size of the rubber washers and of the adjoining steel washers. At the bottom the ⅜-in. pipe is threaded both inside and out and fitted with a plug and coupling. The plug prevents water from getting into the hole below. On the coupling rests a steel washer and above this the lower seal, consisting of several rubber washers made of such rubber as is contained in the inner tubes of automobile tires, except that it should be thicker if this is obtainable. These are followed by another steel washer and then a section of 1-in. pipe 12 in. long and perforated. The ⅜-in. pipe inside is also perforated. Above this is the upper seal, which is identical with the lower seal.

The lengths and arrangement of the piping as shown were found greatly to facilitate the manipulation and extension of the device. Special attention is called to the locking washer, as this arrangement not only prevents the ⅜-in.

pipe from dropping into the hole when adding or removing sections of pipe, but also when moving the device up and down in the hole it transfers the weight to the outer casing. Were this locking washer omitted, the entire weight of the outer casing would be carried by the end coupling on the 3/8-in. pipe. This would cause the expansion of the rubber washers forming the upper and lower seals, which in deep holes would be so great that it would be almost impossible to move the device either up or down.

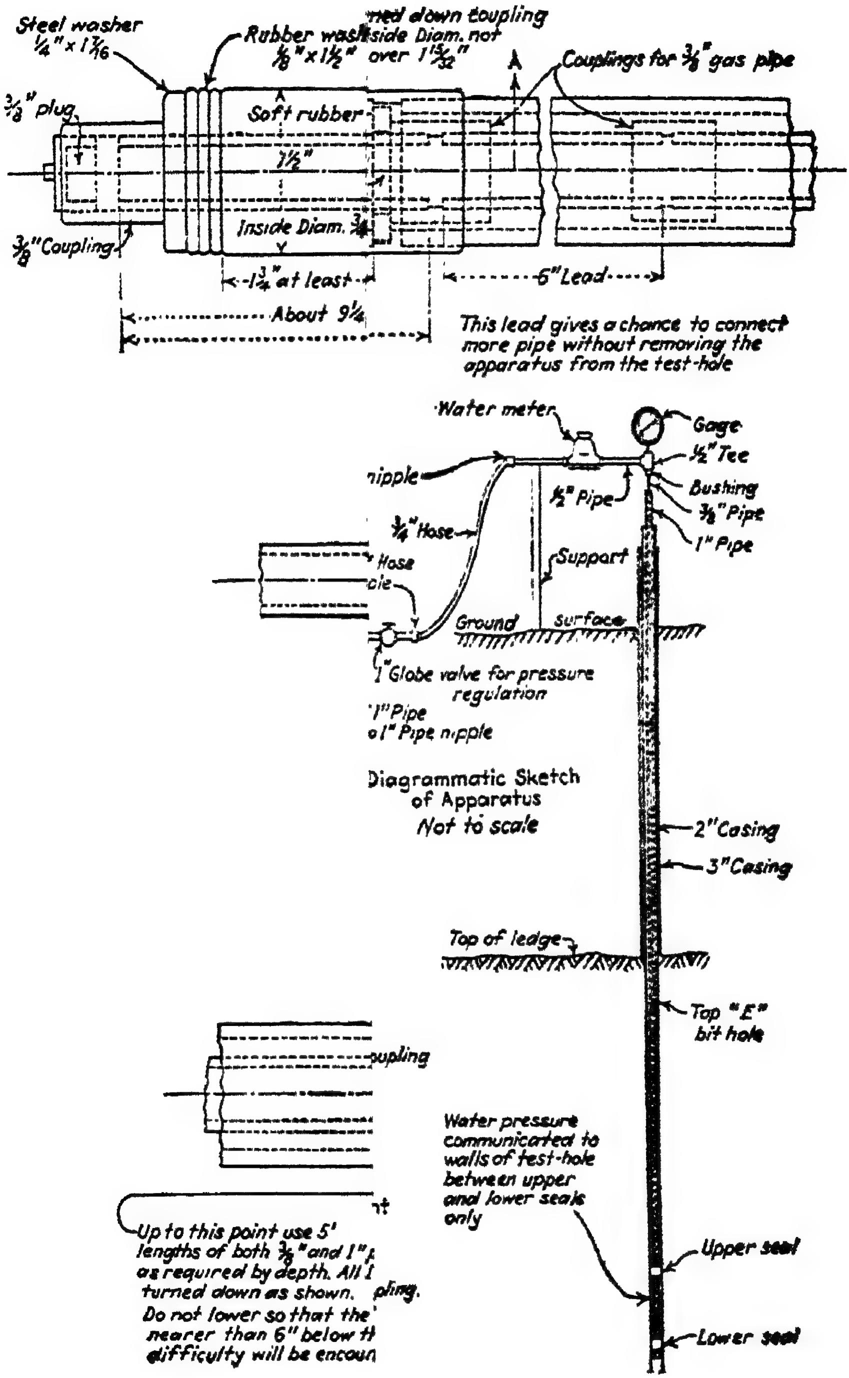
45. Method of Operating Pressure-Testing Device. As shown, a water-pressure gage is located at the top of the 3/8-in. pipe. The water supply is obtained from a pump operated either by hand or by power. An ordinary water meter is inserted in the line and the number of cubic feet of water which the different seams take is recorded as well as the time and the pressure at which the water is taken. The hole should be explored throughout its entire length. Starting at the bottom, the nut on top of the device should be tightened, thus expanding the rubber washers and producing a seal. Then the water pressure is turned on at 25 lb. The elevation of the seal, the water pressure, and the cubic feet of water taken within a 5-min period are recorded. The procedure is then repeated with a water pressure of 50 lb to the sq in. The seal is then loosened, the device raised 1 ft, and the procedure repeated, and so on up to the top of the hole. Often the testing of a hole may be abbreviated by first making the test at the top with the plug removed from the bottom of the 3/8-in. pipe, thus testing the entire hole at one time. If this shows that there is practically no leakage, it is, of course, unnecessary to explore the hole foot by foot as outlined above. The results should be recorded on the graphical record of the hole, thus showing the location, elevation, and capacity of all open seams. The device as described was arrived at only after considerable experience in testing rock foundations ranging in composition from the softest sandstone to mica schist, trap, and granite; and it is believed to be suitable for use under almost any conditions which are apt to be encountered. Precaution must be taken not to apply so much pressure as to injure the rock strata.

46. Periscopic and Feeler Inspection of Drill Holes. In drill holes 6 in. or more in diameter it is possible, by the use of a light bulb and a mirror in the end of a pipe lowered into a hole, to visually inspect the walls of the hole from the top with the aid of a telescope. By adding a movie camera to the setup it is possible to make a continuous photographic log of the walls of the hole.[11]

For exploring and logging diamond drill holes at Norris Dam, feelers were also used. The two arms of the device were scraped against the walls of the hole. When a seam was encountered the arms spread into the opening, causing a bulb at the surface to glow until the arms were again retracted by the further movement of the cable.[12]

[11] "Thousands of Holes Grouted Under Norris Dam." *Eng. News-Record*, Nov. 21, 1935, p. 699.

[12] "Unique Devices Developed to Aid Dam Foundation Grouting," *Eng. News-Record*. Aug. 8, 1935, p. 191.



Diagrammatic Sketch of Apparatus
Not to scale

47. Rough Methods of Determining Suitability of Materials. In Chapter 16, tests of materials which it is desirable to make in connection with the investigation of dam sites, as well as during the period of design and construction, are discussed. However, when one is making a reconnaissance investigation of a proposed site for a dam the facilities of a well-equipped laboratory are often not readily available. Nevertheless it is necessary by a process of elimination to select the most promising locations for the dam site, borrow pits, or quarries before adopting intensive methods of investigation. By observation and very simple rough tests, it is practicable to determine in an approximate and tentative manner the suitability of material for a concrete dam, a rolled fill dam, or a hydraulic fill dam.

If ledge rock is exposed at or near the site and the rock is hard, does not shatter readily under the hammer, and does not show evidence of easy weathering and there is no evidence of local solution, the indications are favorable for a suitable foundation and justify more precise investigation and tests.

If in looking for a quarry site for concrete aggregate or riprap, one finds an exposed cliff where the rock is hard and firm and does not tend to break into thin laminations when pounded with the hammer and there is no evidence of easy weathering, the indications are favorable for a rock suitable for concrete aggregate and for riprap, and the further investigations and tests discussed in Chapter 16 should be made.

If the sand and gravel is well graded and appears to be largely quartz or fragments of hard igneous or metamorphic rock it is probably suitable for concrete aggregate, although if dirty it will have to be washed. Chert (amorphous quartz) is not objectionable provided it is hard.

Such sand and gravel would also be suitable for the pervious portion of an earth dam provided they did not contain too much very fine material.

If sand and gravel when dropped in a bucket of water and sloshed around, leave the water very muddy, the material will probably have to be washed before it will be suitable for concrete aggregate.

If a cohesive borrow-pit material intended for the impervious portion of an earth dam is taken in the hands and kneaded and then rolled out to a diameter about that of an ordinary pencil and shows up just slightly crumbly, then the moisture content is not too great to prevent proper compacting by the use of suitable equipment. The above is merely the roughneck form of the plastic limit test. (See Chapter 16.) It is usually easy to add water to such material if the material is not sufficiently moist, but too high a moisture content in such material may be serious.

If the material consists of sand and gravel with sufficient clay or rock flour to make it sticky when wet, and if when packed and dished it holds water for a long time, it will be suitable for the impervious portion of a rolled fill dam.

Practically all sandy and gravelly materials are suitable for hydraulic fill dams provided only that the fine material included does not contain too much colloidal material. (See Art. 3, Chapter 16.)

To determine probable suitability for hydraulicking, take a small quantity of the fine material mentioned above, and dry it at about 220° F. When dry, powder up the caked portion between the fingers and drop some of it into several dry test tubes to a depth of about 1 in. Shake down well by tapping test tubes with finger. When the sample fails to contract any more, take a paster and put it around the test tube to mark the top level of the dry material. Then fill test tube about two-thirds full with water. Put the thumb over the end and shake until all the sample is in suspension. Then allow the test tube to stand erect and observe it from time to time. If at the end of 24 hr the material has settled through the water to a point showing a volume of not over one and one-half times the dry volume (indicated by the paster) the material will probably prove entirely suitable for hydraulicking for the core of a hydraulic fill dam. If, on the other hand, the material when settled indicates an expansion in volume of two and one-half to three and one-half or more times the dry volume, it would probably behave in a similar manner in the core of a hydraulic fill or semi-hydraulic fill dam and its use in large quantities should be avoided or provision should be made for wasting the finer portion of it. Materials which expand as much as this are usually high in colloids, and continued observation of the test tubes will usually show very little contraction in many weeks.

A powerful field microscope is useful in helping to determine in a qualitative manner the suitability of some materials. Such microscopes, having a magnifying power of 200 diameters or more, are now obtainable. They are made to collapse into a small container which may be readily carried in the pocket. At 200 diameters a particle of rock flour having a diameter of 0.005 mm looks like a sizable rock. Particles which at this magnification look gluey and have no definite crystalline form are probably largely colloidal and have diameters of less than 0.002 mm.

By rough field tests and observations, such as the above, an experienced engineer can usually determine in a tentative manner the suitability of the available materials. As the choice of materials and sites narrows down, the laboratory tests discussed in Chapter 16 should be applied more intensively.

48. Bibliography:

WARREN J. MEAD, "Engineering Geology for Dam Sites," *Second Congress on Large Dams*, Washington, D. C., 1936.

IRVING B. CROSBY, "Engineering Geology Problems at Conchas Dam, New Mexico," *Trans. Am. Soc. C. E.*, Vol. 105, 1940, p. 581.

R. F. LIGGET, "Geology and Civil Engineering," paper presented to Montreal Branch of Eng. Inst. Canada, January 18, 1934.

MORRIS and PEARCE, "A Concrete Gravity Dam for a Faulted Mountain Area," *Eng. News-Record*, December 27, 1934, p. 823.

I. E. HOUK, "New Design Features in Willwood Diversion Dam," *Eng. News-Record*, October 27, 1927, p. 660.

"Unique Devices Developed to Aid Dam Foundation Grouting," *Eng. News-Record*, August 8, 1935, p. 191.

"Thousands of Holes Grouted Under Norris Dam," *Eng. News-Record*, November 21, 1935, p. 699.

E. R. SHEPARD, "Searching for Foundation Beds by Electricity and Sound," *Eng. News-Record*, August 15, 1935, p. 228.

E. R. Shepard, "The Seismic Method of Exploration Applied to Construction Projects," *Military Eng.*, September-October, 1939, p. 370.

A. T. Parsons, "Geophysical Foundation Study by Explosion Wave Method," *Eng. News-Record*, February 14, 1929, p. 273.

"The Present Status of the Art of Obtaining Undisturbed Samples of Soils," report of Committee on Sampling and Testing, Soil Mechanics and Foundations Division, Am. Soc. C. E., Engineering Foundation, New York, March, 1940.

J. C. Stevens, "The Silt Problem," *Trans. Am. Soc. C. E.*, Vol. 101, 1936, p. 207.

R. G. Hemphill, "Silting and Life of Southwestern Reservoirs," *Trans. Am. Soc. C. E.* Vol. 95, 1931, p. 1060.

CHAPTER 2

THE CHOICE OF TYPE OF DAM

1. General Considerations. The purpose of this chapter is to describe only briefly the adaptability of the different types of dams which have been built and to outline the important characteristics which influence their choice for a particular site. An intimate knowledge of the requirements of the different types, as described in succeeding chapters, is of course necessary.

The usual types of dams may be summarized as follows:

Solid gravity concrete dams.[1]
Hollow gravity concrete dams.
Arched concrete dams.
Earth and rock embankments.
Timber dams.
Steel dams.
Other types.

The choice of the type best suited to a particular location or use is a matter on which experienced engineers differ considerably; it is quite often purely a matter of judgment and experience. However, an intelligent study of the existing conditions and requirements will assist materially in the choice.

Safety, of course, is the first consideration. It is impossible to build with safety some types of dams if certain foundations and other characteristics of the site exist. Consideration of these factors will often decrease considerably the number of possible types from which to choose.

The first cost of the structure, as affected by the availability and price of construction materials and other characteristics of the site, is perhaps of next importance.

The choice of type is often limited by the funds available. It will sometimes be found that the difference in cost between an expensive, permanent dam and an inexpensive structure of adequate safety but of short life and high maintenance charges, if set aside at compound interest, will be more than sufficient to provide funds for the higher maintenance cost and a sinking fund to cover the rapid depreciation of the less expensive type. It may be said, however, that, in general, the most permanent dam will be found to be the most economical, and it is usually adopted for ordinary sites, unless the structure is for temporary use, or if sufficient funds are not available.

[1] Other types of masonry for solid gravity dams are now practically obsolete in this country. The principles of design are the same.

A comparison of the several types of dams follows:

2. Solid Gravity Concrete Dams. (See Chapters 7 to 12.) There is no type of dam more permanent than one of solid concrete, nor does any other type require less for maintenance. It is adaptable to all localities, but its height is limited by the strength of the foundations, the height of those on earth foundations having been limited generally to about 65 feet. For localities where the rock is a considerable distance below the surface, an earth-fill dam has frequently been found to be more economical, particularly when a dam of great height is required, because the earth-fill dam does not have to rest on a rock foundation.

The solid gravity concrete dam, being the safest, according to the popular idea; is the most common of all concrete dams, and consequently it has the advantage of appealing to the public in general.

The difference in first costs between solid and hollow gravity concrete dams depends on local conditions. The solid dam requires less cement per cubic yard of concrete, less form work, less expense in placing concrete, and has no steel reinforcement. On the other hand, the hollow dam requires considerably less concrete per linear foot of dam. The lightest type of hollow dam usually requires only 35 to 40 per cent of the concrete required in a solid dam. For a remote location, where materials of construction are expensive, the hollow dam will usually cost less to build than the solid dam; but, in more accessible locations, comparatively near railroads, where concrete aggregates are convenient, the reverse is likely to be true.

The solid gravity concrete dam will usually cost more than a timber dam. However, this may not be the case if a first-class, rock-filled, timber crib dam is adopted at a site where timber is expensive.

An earth or rock embankment will almost always cost considerably less than any form of gravity concrete dam, if materials for the former are found convenient to the site. Therefore, if conditions admit of an embankment, that type of dam is usually to be preferred. The limitations of embankments will be mentioned subsequently.

Because there is considerably less material in an arch dam than in any other concrete type, it costs much less to construct. However, as will be pointed out later, sites particularly suitable for arched dams are rare.

3. Hollow Gravity Concrete Dams. Most hollow dams have been constructed of reinforced concrete of the buttressed types described in Chapter 14.

Turbines and other apparatus have often been placed within hollow dams, thereby making a saving in the necessary housing for such appliances.

Uplift on the base of hollow dams with their buttresses is practically eliminated. However, the problem of eliminating or balancing uplift on horizontally stratified planes below the base is no different from that for a solid dam. (See Art. 17, Chapter 3.)

An advantage claimed for the hollow dam having an upstream face with considerable batter is that it cannot overturn, as the resultant of all forces, for any depth of water, falls well within the base. However, this advantage is

more fanciful than practical, as neither type, if properly designed, should give any cause for worry in that respect.

Hollow dams, being lighter per square foot of area covered, can, by having spread footings, be made to exert less unit pressure on the foundation than solid dams. For this reason the hollow dam is sometimes adopted where the requisite support for a solid dam is lacking.

Poor concrete used in slender hollow dams is a much more serious matter than when used in solid gravity dams. Some hollow dams have given trouble in this respect, particularly in freezing climates. However, this is a criticism of methods of construction rather than of the type of dam.

4. Arched Concrete Dams. (See Chapter 13.) This type is adaptable when the length is small in proportion to the height, and when the sides of the valley are composed of good rock which can resist the end thrust. Under favorable conditions, it contains less material than other concrete types, and being equally permanent, it is usually adopted where conditions permit. Unfortunately, few sites are suitable for this type of dam.

The weight of arched dams is not counted on to assist materially in the resistance of external loads. For this reason, uplift on the base is not an important design factor.

5. Embankments. (See Chapters 17 to 20.) When plenty of materials are convenient to the site, embankments can usually be built for considerably less cost than any form of concrete gravity dam. The use of this type, however, is often limited by the necessity of providing a more suitable spillway for the passage of floods. It is not safe to allow water to spill directly over the embankment, even if it is well paved, unless the volume of the flood per linear foot of crest is small. Therefore a spillway of more suitable character is a necessary adjunct. Some spillways would require most, if not all, of the available length of the dam, in which case an embankment would be out of the question.

The quantity of seepage through pervious material is inversely proportional to the distance the water is required to travel. An earthen embankment, having the longest base in proportion to the height, is particularly adaptable to sites having pervious foundations.

With proper maintenance, the embankment dam should be as permanent as the best. The necessary maintenance charges become rapidly less as the structure settles into its final position and becomes well compacted, tight, and overgrown with proper vegetation to withstand wash from rains.

Earthen dams possess a distinct advantage in landscape work where it is desired to change as little as possible the natural appearance of the site.

6. Timber Dams. (See Chapter 22.) A timber dam is the ideal temporary type; although when well designed, constructed, and maintained, it may last 50 years or more. Maintenance charges, however, are very high, compared with those for other types.

Timber dams are seldom very tight. In fact, a small leakage is necessary for the proper preservation of the timber. Such leakage is a drawback only when the value of the stored water is high.

This type is often used on soft foundations where concrete dams are out of the question, as a slight settlement, which in the timber dam would be permissible, would, in the concrete dam, be an element of considerable danger.

Owing to a scarcity of funds, a timber dam is sometimes adopted with the intention of utilizing it later as a part of the necessary cofferdam for the construction of a more permanent structure.

7. Steel Dams. Only three steel dams of any size have been built in this country. This type is claimed to be more economical than any type of concrete gravity dam and to have other advantages, as discussed in Chapter 21. However, steel dams already built require anchoring to the foundation, a provision which is possible but not considered good practice for concrete gravity dams.

8. Other Types. Various other types of dams have been designed and built. These include peculiarly shaped masonry dams, the many forms of movable dams, and others. These, however, may be considered as structures of unique character, suitable for special conditions not admitting of comparison in the general sense.

CHAPTER 3

PREPARATION AND PROTECTION OF THE FOUNDATION

I. GENERAL

1. Scope. Practically all the general foundation problems, encountered in all types of dams, are applicable to the foundations for concrete dams. Therefore this chapter will treat of the preparation and protection of foundations in general and of concrete dam foundations in particular. Special details applicable to other types of dams will be discussed with those types.

2. General Considerations. The term "foundation," as used herein, means all of that part of the area under and adjacent to the dam which in any way will affect or be affected by loading, scour, or leakage. Investigations of the foundation for the dam site and the compilation of test and other data upon which to base the design are covered in Chapters 1 and 16.

A good foundation is of ample strength to withstand the weight of the structure and to prevent sliding. It must be tight enough to prevent excessive leakage, uplift must be reduced as much as possible, and discharge from the overflow or outlets must not damage it.

Considerable preparation is always necessary in order to provide the requisites of a good foundation. It is probable that more than 90 per cent of all failures of masonry dams have been caused by faulty foundations. It is of the utmost importance, therefore, that this feature of the design should receive proper attention. It is unfortunate that the designer is not always the builder, as many of the assumptions used in the design will depend on the extent and character of the treatment which the foundation receives. In all cases the designer should prepare the specifications for the construction of the dam and should preferably have supervision over the work.

Shallow overburden is always removed to obtain a rock foundation for a dam. However, where the depth of overburden is excessive, dams up to about 65 feet in height have frequently been built on earth. As the cost of a dam on earth is considerably greater than one on rock, the choice of which to use when the depth to rock is neither very deep nor very shallow becomes an economic problem.

II. TREATMENT OF ROCK FOUNDATIONS

3. The Final Surface of Rock Foundations. Surface rock is usually so badly weathered as to be unsuitable for the support of a dam. Excavation to considerable depths is sometimes necessary before rock of an acceptable nature is uncovered.

In the excavation of rock foundations, it is always necessary to take particular care that good rock directly beneath the blasting charges is not unnecessarily shattered. Specifications often require that the last foot or two of the excavation shall be barred and wedged loose. The proper method will suggest itself to the experienced builder when he bears in mind that no part of the final foundation should be disturbed from its original position and that no strata should be jarred loose.

At the Grand Coulee Dam it was specified that no hole for blasting should be drilled more than two-thirds the depth of the proposed excavation which remains to be done.

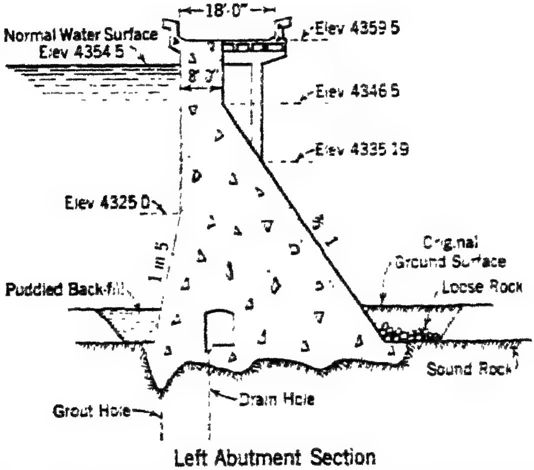


FIG. 1. American Falls Dam, Snake River, Idaho (Ref. 6, Art. 40).

There should be as much resistance to sliding below the surface of the foundation as at the surface. If, therefore, an otherwise good foundation of firm rock contains loose horizontal or nearly horizontal strata on which there is danger of sliding, the excavation should be deep enough to obtain a "toe hold," as shown in Fig. 1, in order that the weight of sufficient bedrock downstream from the dam may be available to resist the sliding forces. The shoulder, affording the toe hold, is frequently excavated by line drilling unless the rock is very good.

In the case of soft, uncemented shales having weak, nearly horizontal bedding planes or other types of weak foundations the toe hold may not be considered strong enough to provide sufficient assurance against sliding. In such cases, a deep heel trench, below the upstream side of the dam, excavated by line drilling and broaching, is provided to be filled with reinforced concrete securely anchored to the dam, as in Fig. 2.

In order that the masonry of the dam shall have the maximum possible bond with the foundation, it is necessary that the final rock surface should be absolutely clean. The thinnest film of dirt of any character on the surface of the

rock is capable of destroying adequate bond and may defeat the entire purpose of the cleansing operations.

There is nothing better for cleaning rock surfaces than jets of a mixture of air and water under considerable pressure. Such a jet is also adaptable to cleaning out vertical seams and pot holes of considerable depth. Such seams should be well plugged with mortar. For concrete dams, it is often specified

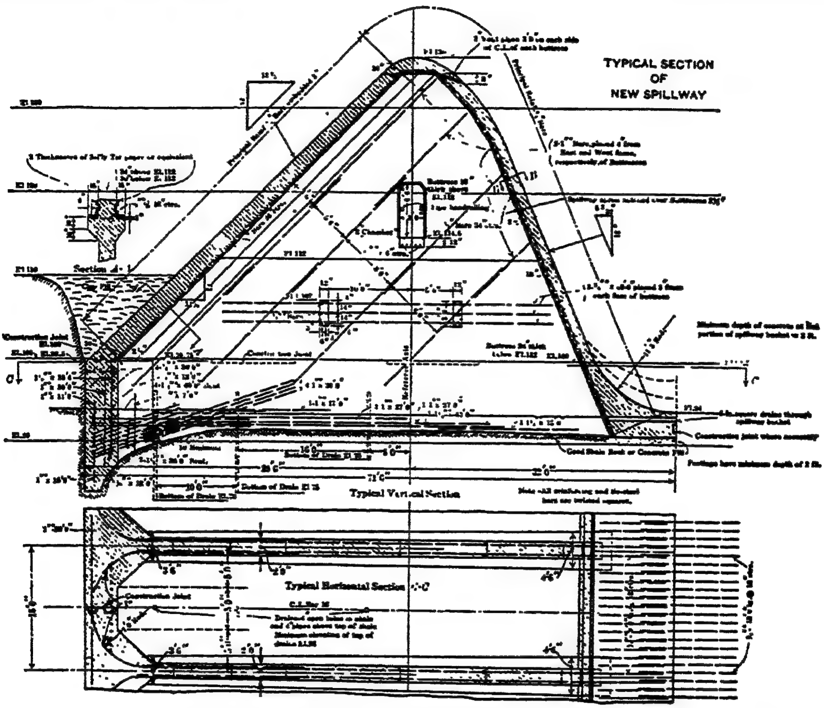


Fig. 2. Stony River Dam (F. W. Scheidenhelm, *Trans. Am. Soc. Civil Engrs.*, 1917, p. 907).

that the finished foundation shall be covered with an inch of rich mortar immediately before the concrete is poured.

The presence of pools of water while placing concrete should be rigidly guarded against.

There should be no large overhanging faces in the rock foundation, and vertical faces should be avoided, except where they coincide with vertical building joints, as they tend to cause shrinkage cracks and excessive shearing stresses.

4. Treatment of Rock Foundation Defects. Most unweathered cemented rocks possess sufficient strength to support dams of usual height. However, special consideration should be given to rocks in which seams or faults and weathered or crushed zones have resulted in separated or partly separated

foundation blocks which might move slightly as a whole under the load of the dam.

Narrow seams and faults frequently can be washed out and grouted as indicated in Art. 7. For wide seams the gouge, weathered or broken rock, or other material which fills them, can be excavated and the seams refilled with concrete. When such defective material lies in a nearly horizontal plane below the surface of the completed excavation, it is sometimes more economical to reach it by a vertical shaft or a large-size drill hole (see Chapter 1, Art. 41), clean out the seam in drifts and fill with concrete rather than excavate the firm rock above it. Cavernous rock and solution channels may also be treated in this manner.

For low dams, small areas of relatively weak rock are sometimes left in place on the assumption that the dam will span over them. In several cases, vertical transverse faults of considerable size have been cleaned out and filled with concrete for a depth only sufficient to provide an arch to span the opening, care being taken of course that the excavating or grouting extends far enough to obtain a tight cutoff at the upstream side.

Where rock, such as uncemented shale, tends to disintegrate when exposed, the final trimming should not be done until just before the concrete is to be placed, since otherwise the rock might dry out and when again saturated by the water from the concrete might form a layer of mud between the foundation and the dam, offering no bond and little resistance to sliding.

In extreme cases, where the rock disintegrates by slaking quite rapidly, the final foundation, as soon as uncovered, should be coated immediately with a bituminous or asphaltic water-proofing material. This procedure is not only of greatest importance in soft shales, but also adds to the strength of the bond between the shales and the concrete.

5. Leakage Through Rock Foundations. Some seepage or leakage through rock foundations is to be expected, as bedrock is seldom if ever entirely unbroken. The main objection to leakage through rock foundations is waste of water and an objectionable appearance, except where it occurs through intermediate layers of clay or sand which might be washed out. Although not necessarily conclusive, leaky foundations have been associated with excessive uplift.

In order to confine the leakage to a reasonable quantity, it is necessary, with poor foundations, to provide a cutoff or artificial impervious barrier under the heel of the dam. For rock foundations there are two general types of cutoffs: first, a trench filled with concrete, and second, holes drilled at frequent intervals and grouted under pressure.

The first type is much to be preferred if it can be constructed at reasonable cost. Before the use of grouted cutoffs became common, the concrete cutoff was sometimes carried in exceptional cases to depths of 50 feet or more. While its advantages are obvious, it has been found that the grouted cutoff is usually sufficiently effective and costs much less.

Cutoffs also tend to reduce uplift on the base of the dam, as indicated here-

inafter under "Earth Foundations." Drainage holes, located downstream from the cutoff, described later, tend to increase leakage slightly.

The general theory of uplift is described in Art. 5 of Chapter 7. The determination of the hydraulic pressures within the foundation, the direction of seepage, the action of drains, and the influence of cutoffs are described in Art. 14 and succeeding articles on Earth Foundations.

6. Grouting Rock Foundations. Grouting of rock foundations is both for tightening to prevent leakage and reduce uplift and for consolidating seamy and broken foundations to make them stronger. This article will treat with methods for tightness, after which Art. 7 will describe consolidation grouting.

Grout is composed of a mixture of neat cement and water, infrequently with certain admixtures as explained later. Asphalt grouting for cavities containing running water is a special case and will be discussed subsequently.

Grouting procedure should not be approached blindly. A thorough study of the geologic features, including the location of faults, solution channels, and areas of weakness by preliminary drillings, described in Chapter 1, lends much assistance to a successful job and, in some cases, is indispensable.

For a cutoff or "curtain" grouting, the required depth of the deepest holes naturally depends upon the nature of the rock in the foundation. The general rule that the deepest grouting should extend to a depth below rock surface equal to one-fourth of the hydrostatic head above rock surface is too inaccurate for serious consideration. The only accurate determination of required depth is a water-pressure test to determine leakage in segregated zones at different elevations.

The tests for each hole consists of pumping clear water into the hole at fixed pressure and measuring the discharge. (See also Art. 44, Chapter 1.)

For rock foundations that do not tighten with depth, the reader is referred to Art. 15, which covers cutoffs in earth and which indicates the futility of deep cutoffs not reaching impervious material.

Curtain grouting is accomplished by grouting one or several lines of holes at the upstream face of the dam. More than one line of holes is not necessary unless rather high pressures are required for the deep holes, in which case several other parallel lines of shallow holes may be required to consolidate the surface over a wider area.

Where vertical seams are present, it is necessary to angle the grout holes in order to intersect all of them. Frequently adjacent holes are angled in opposite directions, in the plane of the grout curtain, forming a crisscross arrangement.

The simplest case for a grouting program is for a curtain having all holes at uniform depth. This is frequently adopted for low dams and also for high dams where the permeability of the rock is relatively uniform above a given stratum and much tighter below that stratum. In this case a "split spacing" method is adopted. A primary series of holes is drilled 15 to 25 feet apart, depending upon the nature of the rock, and then tested with water, and grouted. A second series, consisting of an equal number of intermediate holes, is then drilled, tested, and grouted. A third series is then drilled, tested, and

grouted, thus reducing the spacing to a quarter of that for the first series. The result of a test of any hole is considered an indication of the relative tightness of the foundation between two adjacent holes previously grouted. The process therefore is continued until satisfactory tightness is obtained.

Fig. 2a shows the results of such tests at the Lahontan Dam. The holes there were so close together that they were staggered in two rows, 2 feet apart. However, it was found necessary to grout very few holes in the second row.

Where the foundation contains weak, horizontal bedding planes that are apt to be lifted by grout pressure, as hereinafter explained in more detail,

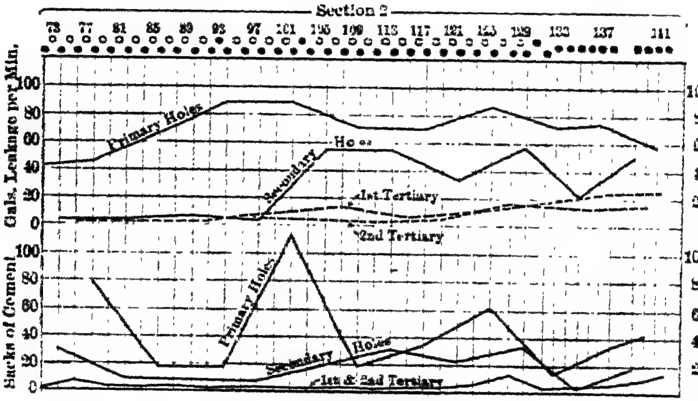


Fig. 2a. Gallons of leakage observed and sacks of cement necessary to grout different borings in Lahontan Dam foundations. (*Eng. Record*, Vol. LXVII, p. 349.)

and where high pressures are required for the lowest depths, "stage grouting" is required. In this method, a hole is first drilled to a shallow depth, grouted at low pressure, and cleaned out before the grout has set up very hard. After the grout in the rock has set, the hole is drilled deeper, grouted under higher pressure, and the process repeated until the final depth is reached and the hole is grouted under the highest pressure.

Another method is to drill the hole to the final depth, extend the grout pipe to the top of the lowest or high-pressure zone, use an expanding seal at the end of the grout pipe to prevent the grout from rising, and grout at high pressure. The pipe is then raised so that the seal is at the top of the next zone to be grouted at somewhat lower pressure. The process is repeated until the highest zone of grouting is reached at the surface. While this method eliminates the drilling out of holes previously grouted, the operation of the seal is troublesome and there is always danger of the high pressure grout bypassing the seal.

As an extra precaution in the stage grouting method the expanding seal has also been used at the top of each zone being grouted, to prevent the higher zones previously grouted from being subjected to the higher pressures.

Usually the tightness of the foundation increases with the depth, and it is not necessary to carry all holes to the deepest depths. In such cases, one or

more lines of shallow holes, from 10 to 20 ft deep, are drilled and grouted first, in order to consolidate the surface and to prevent leakage of high-pressure grout to be used at lower elevations. Then another single line of holes of wider spacing is drilled and grouted at higher pressures. Then a single line of holes constituting a third series of deeper holes of even wider spacing and higher pressures is completed, and so on until the final depth is reached where the spacing is greatest and the pressure is highest.

Although condemned by some engineers on the ground that chips and dust will be crowded into the seams and prevent a flow of grout, shallow grout holes up to about 40 ft have been drilled by percussion drills. However, for deeper holes, core drilling is always used. The former start with a size which will gradually reduce to about 1½ to 2 in. Core drills are usually 2 in. In some projects, some of the last holes are drilled 5 or 6 in. in diameter for visual inspection of the results of grouting by the use of the periscope, which is described in Art. 46, Chapter 1.

The upper end of all holes to be grouted should be provided with threaded pipes, which can be connected to the grouting machine. These pipes must be anchored or weighted to prevent a blowout during the process of grouting. This is sometimes done by cementing the connecting pipes into the drilled holes, or placing them in a concrete cutoff carried far enough into the rock to provide ample grip The drilling, in the latter case, is usually done through the pipes. When it is desired that the drilling shall not interfere with the erection of masonry, the pipes may be carried up along with the masonry and the operations of drilling, testing, and grouting conducted from whatever elevation the masonry has reached, or from a gallery in the dam.

It is preferable to grout each hole before the adjacent holes are drilled, as otherwise the grout, leaking into an adjacent hole under low pressure, may seal it, thus preventing subsequent grouting of that hole from penetrating any great distance.

Each hole should be cleaned out thoroughly with air and water before grouting starts. Water should then be pumped into the holes at moderate pressure to clean out some of the seams, to locate surface leaks to be plugged, and to test for the rate of flow.

Thin grout travels farther than thick grout. Hence it is customary to start with a thin mixture of one part of cement to five or six parts of water and gradually thicken the grout as the hole tightens. This procedure not only reaches remoter seams but offers a minimum of disturbance of the natural formation. A mixture of one part of cement to about two parts of water is frequently adopted for the final grouting, but where the rock is loose, a mixture of one part of cement to 0.6 part of water has been used. Where there is difficulty in sealing a seam, sawdust or shavings added to thick grout has been used with success.

Coarse sand by itself should never be used as an admixture for cement grout, since it tends to settle out and separate from the cement. Rock flour approaching in fineness that of cement has been used successfully as an admixture.

Bentonite, consisting of a clay high in colloids, is also adaptable as an admixture or alone. It is found that bentonite to the extent of 8 per cent of the weight of the cement delays setting of the grout. It is also claimed to hold cement and even sand in suspension and prevent them from settling out. The effectiveness of admixtures can be determined readily by laboratory tests.

Puddled clay used as a grouting medium has been found by tests to be suitable to fill void spaces of considerable magnitude. (See Ref. 16 of Art. 40.)

Pressures used for grouting must be carefully limited to that which will not lift or otherwise move any part of the foundation or adjacent structures. On the other hand, the pressures must be as great as is allowable for speedy work and the largest possible coverage.

In order to detect uplift by grouting, Hays (Ref. 13, Art. 40) recommends drilling a hole to the elevation of the bottom of the deepest hole, anchoring a free rod in the bottom of the hole and allowing the rod to project a few inches above the top of the hole and barely touching a yoke anchored to the surface rock. Any uplift of the surface rock will separate the rod from the yoke.

Where difficulty is experienced in adequately grouting the surface layers without lifting them and when it is feared that high pressure deep grouting may lift the foundation, curtain grouting is done from a gallery after the dam is built or partly built. Machines can be obtained to drill vertical or inclined holes in a 5- by 8-foot gallery.

Grouting pressures range from 5 or 10 lb per sq in. at the surface to 1000 lb in deep holes. No general rule for pressures can be given. The rule of thumb so frequently used, that the pressure in lb per sq in. at any elevation should not exceed the depth in feet, can be exceeded greatly in many places. The writer suggests the following equations as a rough guide only, to be supplemented in practice by judgment and observations during grouting.

Letting p equal the allowed pressure in lb per sq in. at an elevation a distance h in ft below the surface, for massive rock

$$p = h + 1.33h\left(\frac{h}{100} + \frac{3\sqrt{h}}{20}\right) \qquad [1]$$

For sound stratified rock

$$p = h + 1.33h\left(\frac{h}{900} + \frac{\sqrt{h}}{20}\right) \qquad [2]$$

For sound stratified rock which has been grouted above the given elevation

$$p = h + 1.33h\left(\frac{h}{400} + \frac{3\sqrt{h}}{40}\right) \qquad [3]$$

These equations have been plotted in Fig. 3, which also shows, for comparison, the aforementioned rule of thumb.

Fault gouges, stiff clay, or mudstone in thin layers are not particularly objectionable, as far as seepage is concerned, but seams containing mud, silt,

or sand may wash out after the reservoir is filled. This latter class of material therefore should be cleaned out of the seams before grouting. For large solution channels it is best to provide access, clean out and concrete, grouting being applied for a final seal.

Seams are washed out by drilling holes at strategic points and washing back and forth between them with compressed air and water until the water runs clear, using the same pressure as will be used for the primary grouting.

Seams and cracks which intersect the surface may leak grout and hence may have to be sealed by raking out and placing concrete or mortar plugs, caulking

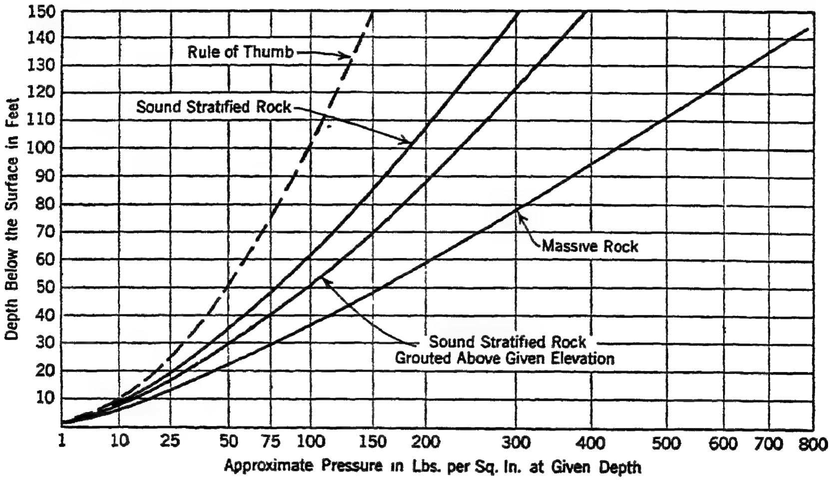


FIG. 3. Rough guide for grouting pressures.

with lead wool, or other means. Bedding planes which intersect the surface are particularly troublesome.

Hot asphalt has proved successful in sealing openings of fairly large size in which there is running water. (See Ref. 20, Art. 40.) The pipes and drill holes are heated by electrically heated wires until flow has started, after which the electrical heat is not required unless the length of hole is great. When the hot asphalt strikes the water it partly solidifies in long strings which stick to the sides of the opening and gradually fill it.

Every effort should be made to plug springs in the area of the grout curtain. If ordinary grouting operations do not close them, they can be allowed to run through pipes until enough concrete has been placed to permit high grouting pressures. Springs below the grout curtain should not be plugged but should be conducted by permanent pipes to tail water.

The method of grouting should be constantly adapted to the job in hand. It is usually found advisable, for each job, to change the method several times during a course of treatment, owing to experience constantly being gained as the work progresses.

Duplex reciprocating grout pumps are much to be preferred to compressed-air grouting machines.

7. Consolidation Grouting. Badly faulted and cracked foundations of otherwise good rock, the solid portions of which might move slightly under the forces acting on the base of the dam, can be successfully consolidated by grouting. The procedure, including washing out of seams, is very similar to that described for curtain grouting, but the required depth is seldom if ever as great.

It is very essential that all seams be thoroughly washed out before consolidation grouting begins. A good example of extensive foundation consolidation grouting for the Norris Dam is given in Ref. 14, Art. 40.

8. Drainage of Rock Foundations. In order to relieve hydrostatic pressure and reduce uplift on the base of the dam and in seams in the foundation, it is frequently advisable, in seamy rock and always for high dams, to drill a line of holes downstream from the previously placed grout curtain to carry away any seepage water which may pass the curtain, as shown in Fig. 1. The drainage holes are connected to a drainage gallery or other means provided to carry the seepage to tail water.

The drainage holes should not be drilled until all grouting operations are completed and should be far enough downstream from the grout curtain to be sure of intersecting only open seams. Water-pressure tests on the drain holes, as previously described for testing grout holes, will check the location of drainage holes relative to the grout curtain.

There is no fixed rule for the determination of the size, spacing, and depth of drainage holes. Experiments on the relative effectiveness of different combinations are badly needed. They vary from 2 to 6 in. in diameter and from 5 to 20 ft from center to center. Their depth depends upon the character of the rock, but a depth equal to one-quarter to one-half the width of the base of the dam is a fair average. There would seem to be no object in extending the drain holes below the depth of grout holes.

Drains of various types, used without drilled drain holes, have been placed between the rock foundations and the dam. However, it seems reasonable to assume that if drains are required at all they will be needed more for the horizontal seams in the rock below the base than at the base of the dam. All drains should discharge above ordinary low tail water, so that the amount of leakage can be observed.

9. Toe Protection for Rock Foundations. Protection of the rock at the toe of spillway dams is frequently needed, particularly if the rock has horizontal stratification. Methods used to reduce the high velocity of spilling water to a velocity which will not erode the rock, are described in Arts. 26 to 39 of this chapter.

III. TREATMENT OF EARTH FOUNDATIONS

10. General Considerations. Concrete dams on earth foundations are numerous; but their use in this country has been limited practically to struc-

tures not more than about 65 ft high for good earth foundations and 30 ft for less resisting earth. This limitation in height may be attributed to the fact that the treatment of earth foundations, to prevent erosion and excessive seepage, is far more expensive than that necessary for rock foundations. In fact, the cost of foundation treatment for dams on earth is often the major part of the total cost of the structure. Consequently, for moderate and high dams it will be found best to adopt another type of structure, or change the site. There have been few precedents for dams higher than noted above, although structurally there would seem to be no reason for a limit to the height, provided sufficient funds are available to meet the unusual expense.

The preparation of the foundation for a dam on earth must be made with five objects in view:

(*a*) To provide ample bearing strength; (*b*) to prevent sliding; (*c*) to prevent excessive seepage under the dam; (*d*) to prevent piping; (*e*) to prevent scouring by the water passing over the dam.

11. Bearing Strength of Earth Foundations. The allowed loads on earth foundations are covered in Art. 4 of Chapter 8. It is essential that there be no excessive settlement in the structure. Unequal settlement is particularly objectionable, as the tightness of the structure is dependent on the absence of settlement cracks.

For hollow dams on earth, the footings are usually spread to reduce bearing stresses. The weight of the hollow Mathis Dike Dam was distributed over the foundation by a mattress covering the entire base, as indicated in Fig. 9, Chapter 14. The weight of solid dams may be distributed through the aprons, which are often reinforced for that purpose. Sometimes bearing piles are required.

12. Bearing Piles. Many types of wood, concrete, or steel bearing piles have been used for excessively weak foundations, the type depending upon the length required and the nature of the foundation materials. The subject of bearing piles is too lengthy for proper treatment here, and the reader is referred to the latest reports of the Committee on Bearing Value of Pile Foundations of the Waterways Division and the Committee on Foundations of the Soil Mechanics Division of the American Society of Civil Engineers.

The use of any type of piling under concrete dams, including cutoff piling and bearing piling, is conducive of roofing. Where such piling is used, the possibility of such roofing should be guarded against by providing filter drains to prevent piping as indicated diagrammatically in Fig. 12, by an upstream blanket to provide sufficient length of path of percolation, or by a combination of these expedients.

Bearing piles should be avoided if a spread base will serve the purpose. Foundations having a dry weight of less than 100 lb per cu ft should be looked upon with grave suspicion. Such loose foundation material generally should be removed.

13. Sliding on Earth Foundations. The dam must, of course, be prevented from sliding as explained elsewhere in this book. Where the coefficient of

friction of concrete on the earth is insufficient, vertical bearing piles, if used, may be counted on to resist the water pressure, although a slight horizontal deflection must be expected. In some cases, considerable expenditures have been made for tests on full size piles to determine such deflections.[1] However, such tests must be made on pile clusters and not individual piles for correct results.

In some dams (see Fig. 4) battered piles have been used effectively in preventing incipient sliding. In the Mathis Dike Dam (Fig. 9, Chapter 14) the

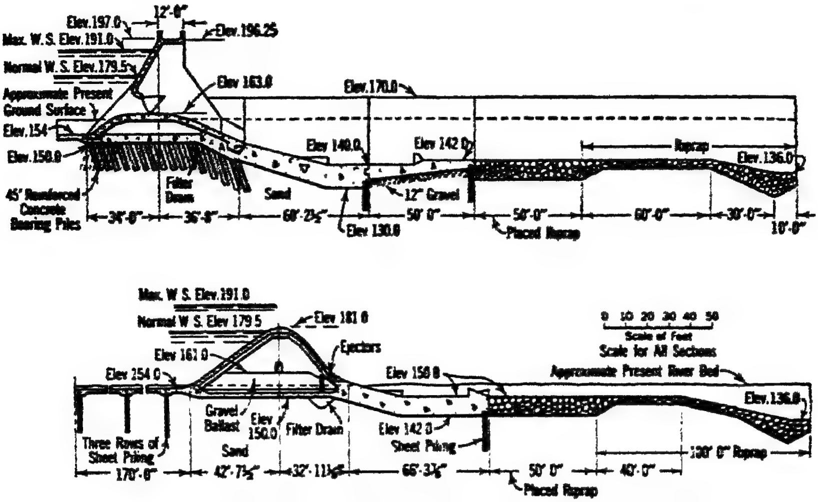


FIG. 4. Imperial Dam, Colorado River, Ariz. (Ref. 6, Art. 40).

longitudinal ribs, at the toe and the middle of the base, are intended to assist in preventing the dam from sliding.

In some cases increased resistance to sliding has been obtained by thoroughly anchoring the dam to a deep cutoff wall at the heel of the structure. Fig. 2 shows such a cutoff for a section of the Stony River Dam where it rested on shale. The same method was used also to prevent sliding where it rested on a clay foundation. The cutoff must, of course, be well tied to the main structure, as shown.

14. The Flow Net. (See also Art. 12, Chapter 17.) The flow net is a diagrammatic representation of the lines of percolation and lines of equal potential in a porous medium, such as earth subject to a head of water. Fig. 5*a* shows a typical flow net for a homogeneous, isotropic foundation without drains or cutoffs. A–B is the length of the impervious elements of the dam. The solid lines are the lines of percolation, or flow lines, and the dotted lines are lines of equal potential. In the true flow net, all of the areas bounded by any pair of flow lines and any pair of equipotential lines are homologous; i.e., have the same ratio of width to length.

[1] L. B. FEAGIN, "Lateral Pile Loading Tests," *Trans. Am. Soc. Civil Engrs.*, 1937, p. 236.

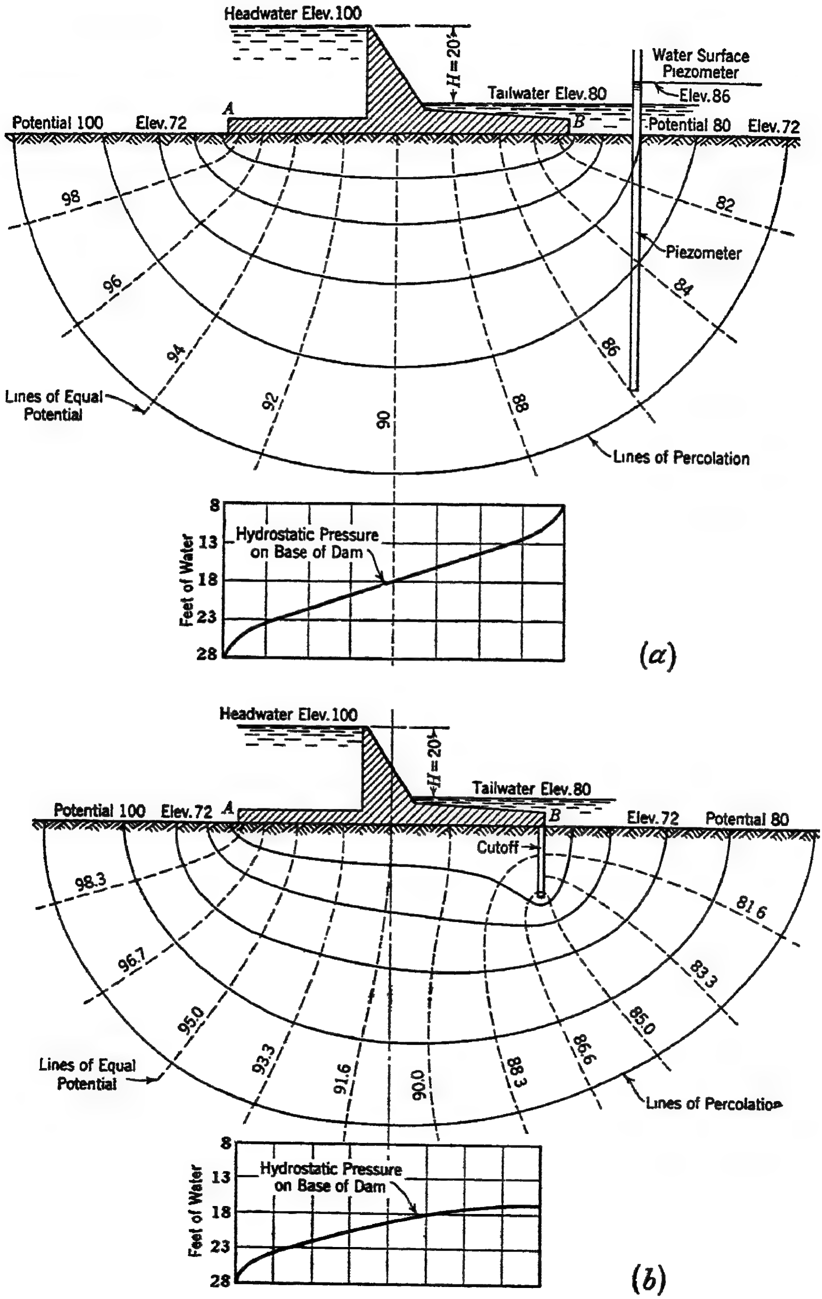

Headwater Elev. 100
H = 20'
Tailwater Elev. 80
Water Surface Piezometer
Elev. 86
A
B
Potential 100
Elev. 72
Potential 80
Elev. 72
98
96
94
92
90
88
86
84
82
Piezometer
Lines of Equal Potential
Lines of Percolation
Hydrostatic Pressure on Base of Dam
Feet of Water
8
13
18
23
28
(a)
Headwater Elev. 100
H = 20'
Tailwater Elev. 80
Potential 100
Elev. 72
A
B
Elev. 72
Potential 80
Cutoff
98.3
96.7
95.0
93.3
91.6
90.0
88.3
86.6
85.0
83.3
81.6
Lines of Equal Potential
Lines of Percolation
Hydrostatic Pressure on Base of Dam
Feet of Water
8
13
18
23
28
(b)

Fig. 5

The potential at any point in the foundation is the elevation to which water would rise in a piezometer at that point, as shown in the figure. For homogeneous, isotropic materials, the flow lines are normal to the equipotential lines. The friction loss of seepage along any flow line is equal to the head, h, on the dam. The head lost in friction between any two lines of equal potential is equal to the difference in their potentials.

The longer the path required for the flow lines, the farther apart will be the lines of equal potential, the smaller the friction loss per linear foot, the slower the velocity and the less the seepage per square foot.

The hydrostatic pressure, in feet of water, at any point on the base of the dam can be determined from the flow net simply by subtracting the elevation of the base from the potential at that point as shown in Fig. 5*a*. One hundred per cent of the hydrostatic pressure is always assumed for uplift for dams on earth.

Should the permeability at all points in the foundation be increased, the amount of seepage and velocity of flow would be increased in direct proportion but the flow net and amount of uplift on the dam would not change.

The flow net can be determined analytically for simple conditions; but, where there are one or more cutoffs, strata of variable permeability, lenses of different permeability, drains, and other complications, it can be made most conveniently by the electrical analogy model test.

Such tests are based on the fact that Ohm's law of flow of electricity is identical with Darcy's law of flow of water through porous materials. That is, in each case, the loss of potential is directly proportional to length of flow and the velocity of flow.

Thus, for two dimensional studies, we have only to establish a flat electrical conductor having the same shape as the cross-section of the foundation and with the relative conductivity of its various parts the same as the relative permeability of the foundation. With one terminal representing head water pressure and the other tail water pressure, the percentage loss of electrical potential between the head water terminal and any point in the conductor will correspond to the percentage loss of hydraulic potential to the corresponding point in the foundation.

Such tests can be made by using a solution of salt, ammonium chloride, or other proper conductor in a shallow tray, with depth of solution varying with desired changes in conductivity. (See Refs. 3, 8, 10, and 12, Art. 40.)

The lines of equal potential having been obtained by this means, the flow lines are drawn in simply to form squares with the equipotential lines.

Flow nets can also be obtained from homologous small sand models, in which piezometers are used to measure the potential at various points. If the sand model is bounded on one side by a glass plate, the lines of percolation can be visualized by inserting dye at places on the surface of ingress of the water. However, the flow net can be constructed if either the lines of percolation or the equipotential lines are given.

It is not necessary to use the same materials in the model as in the prototype as long as the *relative* permeability at all places is the same. The effective size [2] of the finest material used in a sand model should not be less than 0.7 mm to avoid distortion by capillarity and the coarsest should be tested to see if it follows Darcy's law.

Flow through thin plates, so close together that Darcy's law applies, have also been used for model tests. (See Ref. 11, Art. 40.) The lines of percolation can be indicated by inserting dye.

For methods of testing the permeability of the foundation see Art. 22 and other articles in Chapter 16.

Frequently the permeability of sedimentary soil is considerably greater (5 to 15 times) in the horizontal than in the vertical direction. This makes quite a difference in the flow net and makes more danger of piping. The condition cannot be simulated directly in any model yet devised. However, the test can be made by making all horizontal dimensions of the model a percentage, p, of the corresponding vertical dimensions, where p is equal to [3]

$$p = \sqrt{\frac{K_v}{K_h}}$$

where K_v = the coefficient of permeability in the vertical direction, i.e., the rate of discharge through unit area under a friction gradient of unity, and K_h = the corresponding coefficient in the horizontal direction.

After the test is made and plotted, the horizontal dimensions of the model and the flow net are divided by the percentage, p, to obtain the conditions in the prototype.

The flow net should be determined for all important dams where a tight cutoff to impervious material is not possible.

The "shortest path of percolation" is the shortest line of percolation as shown by the flow net. Bligh's and Lane's "line of creep," discussed more thoroughly in Art. 18, is the line of contact of the base of the dam and cutoffs with the foundation. Should the line of creep be materially more pervious than the rest of the foundation, the flow net for a homogeneous foundation would not apply, since the percolation would tend to follow the line of creep. The term, "path of percolation," will be used in a general sense, since those features which tend to influence the path will apply both to homogeneous foundations and to the line-of-creep theory.

Other things being equal, a long path of percolation reduces seepage and the possibility of piping. The effect of the length of the path of percolation on uplift will be explained in Art. 17.

The desired length of the path of percolation can be obtained by an upstream apron, a downstream apron, one or more cutoffs, or a combination of all these, as explained more in detail later. Many types and combinations

[2] That size of which 90 per cent is greater and 10 per cent smaller.

[3] A. CASAGRANDE (after A. F. Samsioe), *Trans. Am. Soc. Civil Engrs.*, 1935, p. 1292.

have been proposed and built. Modern practice in this respect will be discussed later.

15. Seepage Through Earth Foundations. A small amount of seepage through earth foundations for dams is to be expected. Excessive seepage is objectionable, not only on account of waste of impounded water, but princi-

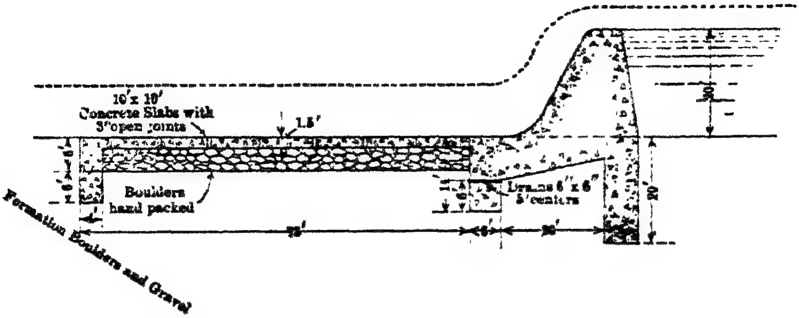


Fig. 6. Granite Reef Dam, Salt River, Ariz. ("*The Design and Construction of Dams,*" *Wegmann.*)

pally because of danger of piping. Seepage is reduced to best advantage by a watertight cutoff to rock or other impermeable stratum. It can also be reduced by increasing the length of path of percolation as previously explained.

In silt-laden streams, deposits in the reservoir bottom filtered out of turbid water will often seal the pores quickly and thus reduce seepage. The Granite Reef Dam, shown in Fig. 6, is founded on gravel and boulders. The length of the path of percolation was only about three times the head on the dam. The considerable leakage when the dam was first used was soon stopped by the large quantity of silt carried by the river.

A cutoff, under a dam, which carries only part way to an impervious stratum is not efficient. Fig. 7, plotted from the results of research tests by the writer and tests by Turnbull, for the Kingsley Dam in Nebraska, shows the relation between seepage and depth of cutoff. Those by the writer were for a ratio of base width of dam to depth of pervious material of 1.4 while the corresponding ratio for the Kingsley Dam was 14.

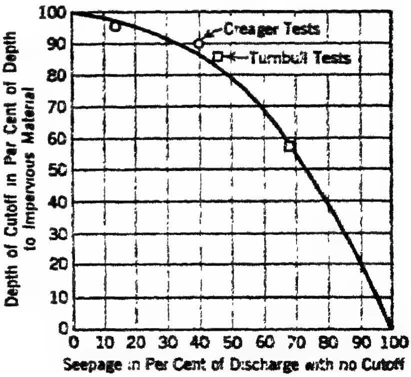


Fig. 7.

Fig. 7 shows that, for a cutoff occupying 90 per cent of the distance to the impervious material, the seepage will still be 35 per cent of the seepage for no cutoff.

If the permeability of the foundation, K, is known, the amount of seepage past any length of dam can be computed from the flow net. Casagrande has shown[4] that the seepage loss in cubic feet per second may be found by the following equation, if the flow net is formed of squares.

$$Q = \frac{LKHs}{60} \qquad [4]$$

when Q = discharge in cu ft per sec;
L = length of the dam considered, in feet;
K = coefficient of permeability of the material in cu ft per min per sq ft of area;
H = head on the dam in feet;
s = ratio of the number of squares between any two neighboring equipotential lines and the number of squares between any two neighboring lines of percolation.

For example, the coefficient of permeability, K, is the discharge in cubic feet per minute per square foot of the foundation material under unity gradient; i.e., when the friction loss per foot of travel is unity.

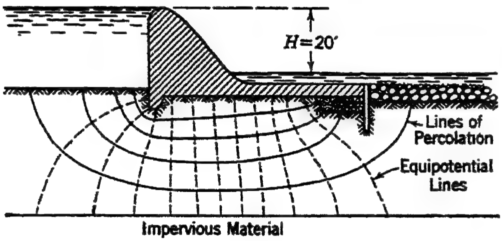


FIG. 8. Example of flow net for seepage calculation.

In Fig. 8 there are 5 squares between any two neighboring equipotential lines and 12 squares between any two neighboring lines of percolation. Therefore

$$s = \tfrac{5}{12} = 0.416$$

Assume that $K = 50 \times 10^{-4}$ cu cm per sec per sq cm,[5]

$$K = \frac{50 \times 60}{10{,}000 \times 30.48} = 0.00984 \text{ cu ft per min per sq ft}$$

and

$$H = 20 \text{ ft and } L = 500 \text{ ft}$$

Then the expected seepage would be

$$Q = \frac{500 \times 0.0098 \times 20 \times 0.416}{60} = 0.68 \text{ cu ft per sec}$$

[4] Op. cit., p. 1290.
[5] Usual laboratory nomenclature.

Without the aid of the flow net, the computation of seepage is difficult. The best method is to search references of Art. 40, particularly Refs. 3, 7, and 8, and other sources for a flow net of the closest applicable conditions and apply Eq. 4. If tests on the coefficient of permeability have not been made, as indicated in Chapter 1, but the approximate nature of the material is known from mechanical analyses, the coefficient can be taken from Table 2, Chapter 16. Great accuracy is not possible since the permeability of different materials, with the same effective size and porosity, vary as much as several hundred per cent.

16. Piping. Piping may be defined as the movement of material from the foundation by the velocity of the seeping water as it issues from the soil below the dam. Formulae have been written defining the velocity required to lift soil particles of different sizes. However, *incipient* piping occurs when the pressure of the seeping water at any point in the foundation, as shown by the equipotential lines of the flow net, is greater than the saturated weight of the soil above that point. Under such conditions the soil becomes supersaturated, quick, and incapable of supporting any load and actual piping is imminent. It is therefore with respect to that condition that the dam should be investigated.

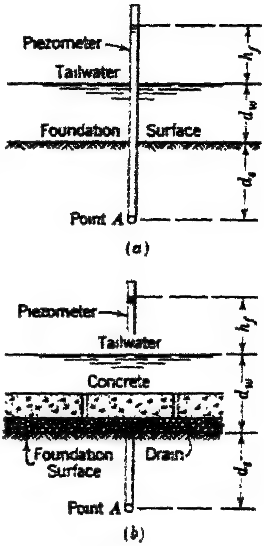


FIG. 9.

Where seepage emerges, all points below the surface of the ground, such as Point A in Fig. 9a, should be investigated for unstable conditions.

The upward unit pressure at Point A corresponds to the height the water will rise in the piezometer, or

$$\text{Upward pressure} = 62.5(d_e + d_w + h_f)$$

where h_f is the friction loss between Point A and the surface of the ground.

The total downward pressure at Point A is equal to the total weight of soil and water above Point A, or

$$\text{Downward pressure} = 62.5d_w + 62.5d_e + wd_e$$

where w is the submerged weight of earth.

For equilibrium, these must be equal, or

$$wd_e = 62.5h_f$$

That is, the submerged weight of earth above Point A must be equal to the force of friction between Point A and the surface of the ground.

For an approximate weight of submerged earth of 62.5 lb per cu ft

$$d_e = h_f$$

The factor of safety against piping is

$$S = \frac{d_e}{h_f} \tag{5}$$

Because h_f/d_e is the friction gradient and h_f is also equal to the loss of potential between Point A and the surface of the ground, Eq. 5 may be stated in two ways, both of which are used by designers.

(1) The friction gradient between any point in the foundation and the surface of the foundation must be less than unity.

(2) The loss of potential between any point in the foundation and the surface must be less than the distance to the surface.

The foregoing analysis applies to the condition where there is no concrete or other material on the surface of the foundation.

When the concrete of the dam or downstream apron rests directly upon the soil, it is simply a matter of balancing the uplift pressure, as explained in Arts. 14 and 17.

When there is a filter drain under the dam or the downstream apron, the uplift on the base at the drain is zero, but every point in the foundation must be investigated for incipient piping as heretofore explained. In other words, drains to prevent piping must themselves be weighted down.

Fig. 9*b* represents typical conditions to which the analysis previously given for Fig. 9*a* will apply strictly, provided the submerged weight, $\overline{W}$, of the drain and concrete is considered. The resulting equations for security against incipient piping, that is, the soil below the drain becoming quick, are

$$S = \frac{d_e + \dfrac{\overline{W}}{62.5}}{h_f} \tag{5a}$$

The authors recommend a factor of safety of $S = 4$ for thoroughly explored homogeneous foundations, where the ratio of horizontal to vertical permeability is well determined and the foundation subjected to a good model test. This factor of safety must be increased according to judgment, depending upon the information or rather lack of information on the characteristics of the foundation and upon whether or not a model test has been made, particularly where the foundation contains stratifications and lenses of relatively pervious materials. Factors of safety of 10 or more are not too severe under bad conditions.

However, it will be shown later that, with the proper use of controlled drainage, these factors of safety are easily obtained.

It has been explained previously that the closer the lines of equipotential, the higher the seepage velocity and the greater the loss of potential per linear foot. For the simple case of Fig. 5*a*, used here for explanatory purposes but not as a recommended design, it will be noted that the lines of equal potential are closer at A and B than elsewhere. This is true in every case where there is

an abrupt corner about which the water must flow. Therefore, at such places the rate of loss of potential and velocity of seepage is relatively very great. Theoretically it is infinite exactly at the point of turn.

This is a matter of no concern at the entrance surface nor in the foundation where this condition cannot move the material, as the material has no place to go. However, it is conducive to piping where the seepage emerges below the dam.

In Fig. 5*a* the high velocity at *B* would assuredly cause piping.

These theoretical conditions would be improved, as far as piping is concerned, by the addition of a cutoff at the downstream end, as shown in Fig. 5*b*, which would transfer the high seepage velocity to the bottom of the cutoff.

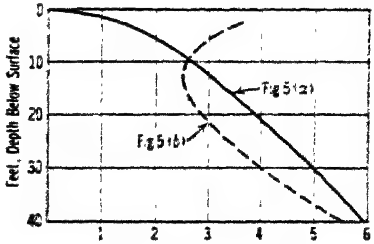


FIG. 10.

However, note that for Fig. 5*b*, the downstream cutoff has increased considerably the uplift pressure on the downstream apron and, for that reason, is objectionable. On the other hand, where the later recommended controlled drainage is not used, some depth of cutoff at the toe, to reduce surface exit velocity, is absolutely necessary.

Fig. 10 shows the relative safety of the two designs. For Fig. 5*a* the factor of safety against piping is zero at the surface of the ground and increases with depth below the surface. For Fig. 5*b* the minimum factor of safety is 2.5 at about 13 ft below the surface and is greater elsewhere.

The cutoff of Fig. 5*b* is also of benefit to protect the overflow dam from scour at the toe and should, of course, be extended to a point well below possible scour, such possible scour being included in the model test.

17. Uplift. As indicated in a previous chapter, the uplift pressure on the base of the dam must be considered in the determination of the stability of that structure. The determination of the amount of uplift from the flow net is indicated in Art. 14. If an analogy model test is not made, the uplift to be taken care of should be determined from a comparison of the flow nets of similar designs as indicated in Art. 15. An ample factor of safety should be used where the foundation has not been thoroughly explored and lenses of different permeability or other heterogeneous conditions may exist.

Not only must uplift at the base of the dam and apron be considered, but also at deeper points in the foundation. For instance, a dam or apron under which is an adequate filter drain, would have no uplift on its base, but there is still hydrostatic pressure at points a few feet below the base. In extreme cases, as mentioned before, this hydrostatic pressure might be capable of lifting the saturated weight of earth above it and the weight of the apron and water above the apron. In other words, all filter drains must be adequately weighted. Any uplift on the apron below the dam must be balanced by the weight of the

apron and water above it. However, the force of the spilling water may reduce the depth of water over the apron as explained in Arts. 26 to 39 of this chapter.

The use of a downstream apron to increase the length of the path of percolation will increase uplift on the dam. A cutoff at the upstream end of the dam or an upstream apron will reduce uplift. Weaver (Ref. 8 Art. 40) has deter-

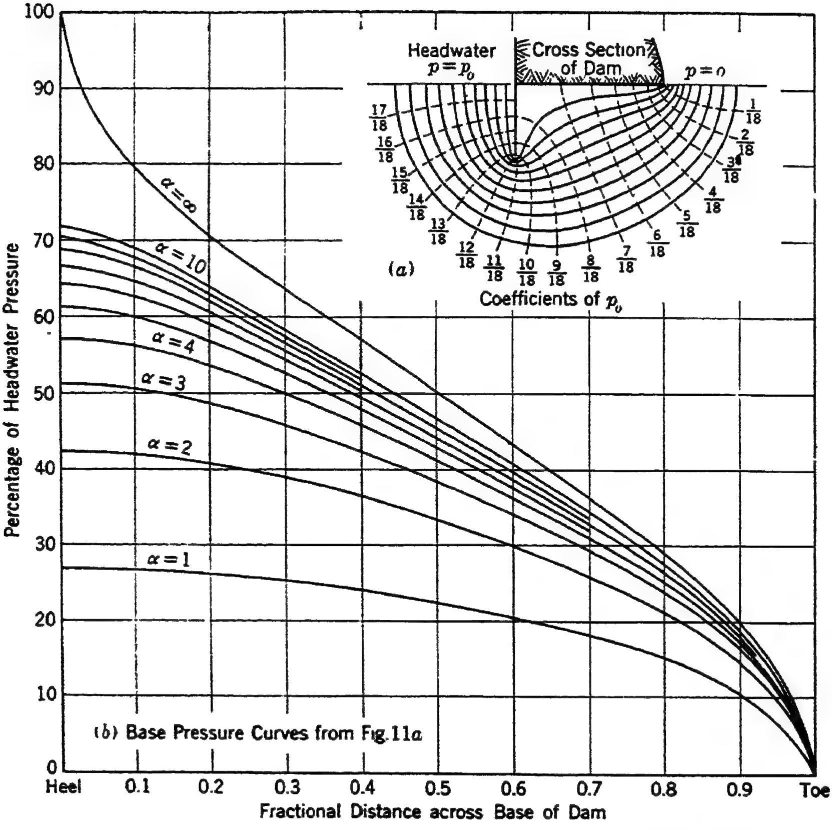


FIG. 11. Flow net for heel cutoff (L. F. Harza, *Trans. Am. Soc. Civil Engrs.*, 1935, p. 1367)

mined mathematically the data from which Fig. 11 was drawn, to indicate the effect of a single upstream cutoff on uplift for a dam on a pervious foundation of infinite depth without drains. In Fig. 11, α is the ratio of the base width of dam and aprons to the depth of the cutoff. Values of α = infinity correspond to no cutoff and agree with Fig. 5*a*.

A lens of relatively pervious material downstream from the heel of the dam reduces uplift. Such a lens upstream from the heel increases uplift.

The thickness of the downstream apron, subject to uplift, can be reduced if anchored to piles driven into the foundation.

18. Line of Creep. Bligh's line-of-creep theory[6] revised by Lane (Ref. 7 Art. 40) is based on the theory that resistance to percolation along the "line of creep," i.e., along the line of contact of the dam and cutoffs with the foundations, may be less than directly through the foundation materials on account of the difficulty of securing an intimate contact. Thus, according to this premise, the flow will concentrate along the line of creep, causing higher velocities at the exit than would be indicated by a flow net constructed for a homogeneous foundation. This loosening of the contact between the dam and the foundation, caused by unequal settlement and general settlement where bearing piles are used, is called "roofing."

On account of the fact that the exact nature of roofing and disturbance of the contact of cutoffs with the foundation is indeterminate, Bligh recommended a minimum "creep ratio" of length of line of creep to head on the dam, depending upon the nature of the materials in the foundation. His recommended creep ratio was based on the observation of the success or failure of a number of dams.

Lane, adopting the same general theory, and after a study of more dams of this type, recommends using a weighted line of creep, in which horizontal contacts with the foundation and slopes flatter than 45°, being less liable to have intimate contact, are given only one-third the value of steeper and vertical contacts. That is, his line of creep is the summation of all the steep and vertical contacts plus one-third the sum of all the flatter and horizontal lines of contact between head and tail water following along the contact surface of the base of the dam and cutoffs.

Should the distance between the bottoms of two cutoffs be less than one-half the weighted creep distance between them, twice the distance between them should be used instead of the actual line of creep between them.

Lane's recommended weighted creep ratios, i.e., the ratio of weighted creep distance to head, are given in the first column of Table 3, subject to certain modifications for special conditions.

Lane's line-of-creep method has not been universally accepted exactly as recommended. The principal points of disagreement have to do with those dams which have been designed by the aid of model tests to determine flow net analyses and with ample provision for taking care of seepage with filter drains to prevent piping.[7]

The authors offer the following analysis which was not covered in the discussion of Lane's paper.

In Fig. 12, AB represents the base of a dam having a cutoff at the upper end, a drain at the lower end, and roofing under the dam. In Fig. 12a the roofing has been indicated by a horizontal line a short distance below the foundation. The seepage is free to flow along this roofing toward the drain. It shows the resulting flow net before any change in the foundation occurs. Piping into the roofing space is probable.

[6] "Dams, Barrages, and Weirs on Porous Foundations," *Eng. News*, December 29, 1910

[7] See discussion of Lane's paper, Ref. 7, Art. 40.

TABLE 3[1]

Recommended Weighted Creep Ratios

Material	Case *a* (Lane) 100%	Case *b* 80%	Case *c* 70%
Very fine sand or silt	8.5	6.8	6.0
Fine sand	7.0	5.6	4.9
Medium sand	6.0	4.8	4.2
Coarse sand	5.0	4.0	3.5
Fine gravel	4.0	3.2	2.8
Medium gravel	3.5	2.8	2.5
Coarse gravel, including cobbles	3.0	2.4	2.1
Boulders with some cobbles and gravel	2.5	2.0	1.8
Soft clay	3.0	2.4	2.1
Medium clay	2.0	1.6	1.5
Hard clay	1.8	1.5	1.5
Very hard clay or hardpan	1.6	1.5	1.5

[1] Tables 1 and 2 not used.

When piping occurs along the line of roofing, material so piped will be carried forward toward the drain, *C*. However, if the drain is provided with a filter, such moved material will be trapped. The result will be a rearrangement of the roofing conditions, as indicated in Fig. 12*b*, tremendously exaggerated as to vertical distances, i.e., the roofing near the drain will be eliminated and that near *A* will be increased.

The flow net of Fig. 12*b* indicates this condition. It must be remembered that the depth of the roofing spaces, indicated diagrammatically in these sketches, is in reality at most a very small fraction of an inch and any readjustment which might take place is inconsequential.

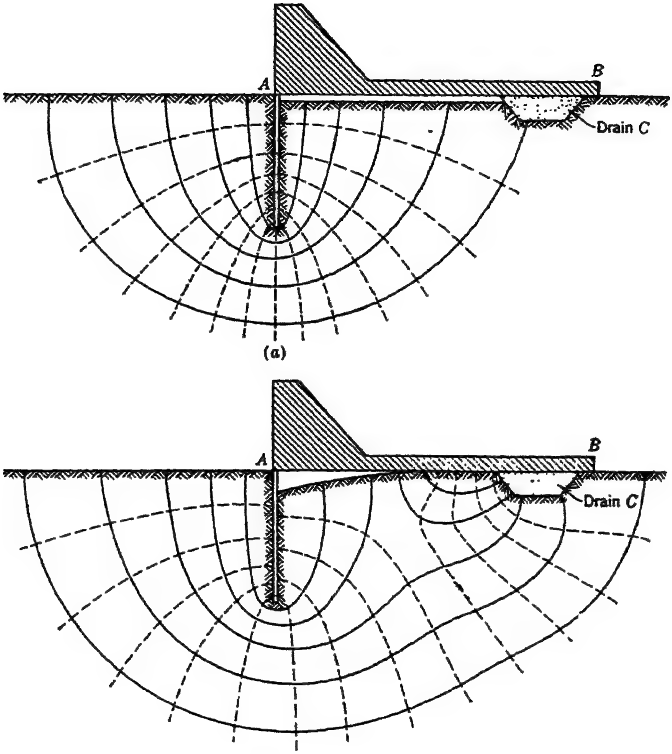
The result is that, since a loss of foundation material under the dam cannot occur, failure from roofing is impossible if the dam is properly protected by a filter drain. The foregoing theory does not include the possibility of piping downstream from the drain. This can be indicated by a flow net analysis.

The authors' opinions, fortified by the opinions of the discussors of Lane's paper and by Lane's closure of the discussion is indicated in Table 3 and explained as follows:

Case a. Although the authors will later recommend the use of filter drains and downstream cutoffs wherever possible and the use of flow net analyses for

important dams it is recommended that, where these provisions are not used, Lane's weighted creep ratios be used, as indicated in Table 3.

Case b. Where drains are properly provided but no flow net analyses are made, use 80 per cent of Lane's weighted creep ratios.



(a)

(b)

Fig. 12. Approximate flow nets.

Case c. Where both drains and flow net analyses are used, adopt 70 per cent of Lane's weighted creep ratios even though the flow net analysis may indicate that a smaller ratio would be safe.

However, for cases *b* and *c*, the weighted creep ratio should not be less than 1.5 under any circumstances.

However, the base of dams and aprons supported on bearing piles should be given a weighted creep of zero if a filter drain is not used as shown in Fig. 12.

In some cases the calculated line of creep may result in a dam through which there is too much seepage. In such cases, provisions to reduce seepage makes for an increase in the line of creep.

19. Recommended Design for Earth Foundations. As mentioned before, many combinations of appurtenances for foundation protection have been used, consisting of one or more upstream cutoffs, upstream aprons or blankets, downstream aprons, one or more lines of downstream sheet piling, and drains of various kinds at various places. An indication of the extreme variance in design is shown by examples of foundation treatment for over 150 dams on earth, contained in Ref. 5 of Art. 40. This reference contains also an excellent bibliography.

So much depends upon the conditions at the site that no general standard is in use. However, Fig. 13 may be used as a basis, variations from which must

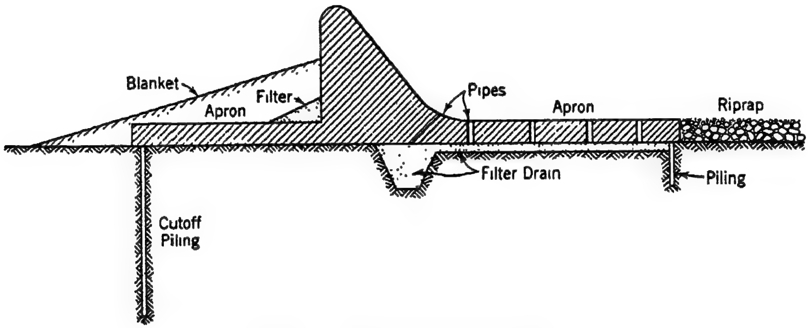


Fig. 13. Diagrammatic example.

be made to suit local conditions. It consists of an upstream apron, a blanket with filter, described later, an upstream cutoff, a main filter drain, a downstream apron, underneath which is a filter drain, and a downstream row of sheet piling. The downstream row of sheet piling is used to prevent damage in case the riprap washes out and to reduce the velocity of residual seepage.

Several cutoffs were used at the Imperial Dam (Fig. 4). However, the writer has found that a single cutoff at the upper end of the upstream apron has proved sufficiently effective with, in other cases, an auxiliary cutoff at the upstream end of the base of the dam to reduce slightly more the uplift on the dam and to serve as a safeguard in case of failure of the apron. The best arrangement can be obtained only with a flow net from an analogy test.

A filter drain was located at the downstream toe of the dam. It was probably proved by model tests to be adequate, since no filter drains are located under the apron. The weighted creep ratio is 8.0 for a sand foundation. Many safe dams, under similar conditions, have been built with much lower ratios.

The Granite Reef Dam (Fig. 6) rests on a foundation of gravel and boulders. The weighted creep ratio when the dam was first built was 2.8, and no piping occurred. However, when the dam was first used, considerable water passed under the cutoff and out through the drains on the apron. This explains the necessity for increasing the weighted creep ratios of Table 3 if necessary to

19. Recommended Design for Earth Foundations. As mentioned before, many combinations of appurtenances for foundation protection have been used, consisting of one or more upstream cutoffs, upstream aprons or blankets, downstream aprons, one or more lines of downstream sheet piling, and drains of various kinds at various places. An indication of the extreme variance in design is shown by examples of foundation treatment for over 150 dams on earth, contained in Ref. 5 of Art. 40. This reference contains also an excellent bibliography.

So much depends upon the conditions at the site that no general standard is in use. However, Fig. 13 may be used as a basis, variations from which must

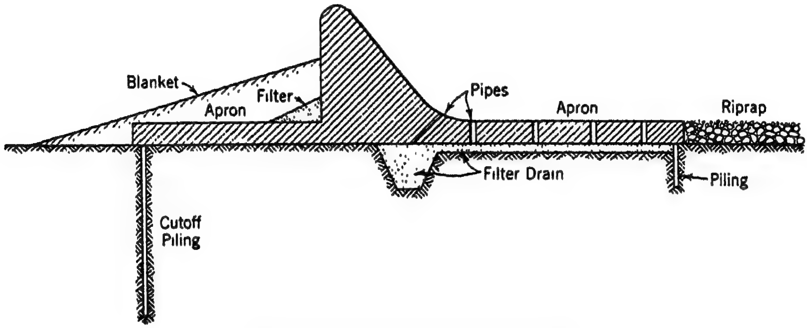


FIG. 13. Diagrammatic example.

be made to suit local conditions. It consists of an upstream apron, a blanket with filter, described later, an upstream cutoff, a main filter drain, a downstream apron, underneath which is a filter drain, and a downstream row of sheet piling. The downstream row of sheet piling is used to prevent damage in case the riprap washes out and to reduce the velocity of residual seepage.

Several cutoffs were used at the Imperial Dam (Fig. 4). However, the writer has found that a single cutoff at the upper end of the upstream apron has proved sufficiently effective with, in other cases, an auxiliary cutoff at the upstream end of the base of the dam to reduce slightly more the uplift on the dam and to serve as a safeguard in case of failure of the apron. The best arrangement can be obtained only with a flow net from an analogy test.

A filter drain was located at the downstream toe of the dam. It was probably proved by model tests to be adequate, since no filter drains are located under the apron. The weighted creep ratio is 8.0 for a sand foundation. Many safe dams, under similar conditions, have been built with much lower ratios.

The Granite Reef Dam (Fig. 6) rests on a foundation of gravel and boulders. The weighted creep ratio when the dam was first built was 2.8, and no piping occurred. However, when the dam was first used, considerable water passed under the cutoff and out through the drains on the apron. This explains the necessity for increasing the weighted creep ratios of Table 3 if necessary to

limit waste of water, as previously explained. For this dam, the flow was soon stopped by silt deposited on the floor of the reservoir.

The Svirstroy Dam,[8] U.S.S.R., Fig. 14, has a cutoff at the upper end of the upstream apron. Since the foundation was clay, this provided sufficient length of path of percolation to eliminate excessive leakage. Extending the filter

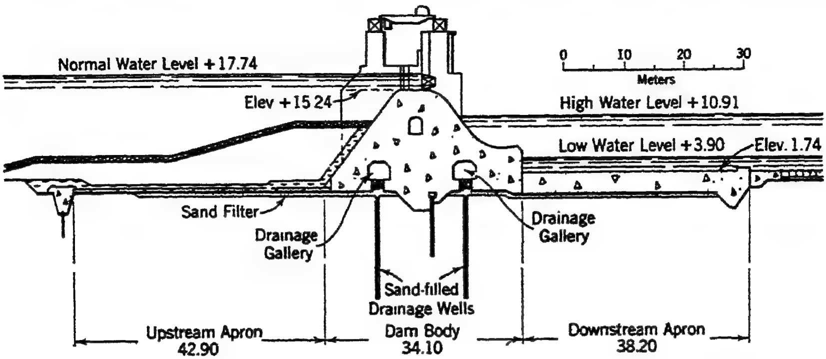


FIG. 14. Svirstroy Dam, U.S.S.R. (*Proc. Intern. Conf. Soil Mechanics*, Cambridge, Mass., 1936).

drain under the upstream apron reduced uplift and therefore provided the greatest net downward water pressure on the apron, and this, since the apron was tied into the dam proper, assisted in preventing sliding.

The Cochiti Dam (Fig. 15) rests on sand, gravel, and cobbles. It has a weighted creep ratio of 5.5., a conservative value. A "selected gravel" drain was placed under the dam where shown.

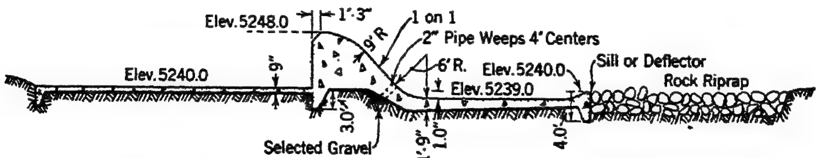


FIG. 15. Cochiti Dam, New Mexico (*Civil Eng.*, Jan. 1933, p. 12).

20. Upstream Apron. The upstream apron is used, of course, to increase the length of the path of percolation. It is better than a downstream apron for that purpose since there is no unbalanced uplift under it to be balanced by weight of concrete.

It may be made of reinforced concrete or it may be simply an impervious earth blanket. An opening of the joint between the apron and the dam, due to unequal settlement, must be guarded against.

In Fig. 13 is shown a filter at the junction of the apron and the dam and an impervious blanket above the apron. Should a crack occur at that place, the

[8] "Some Features in Connection with the Foundation of Svir 3 Hydro-Electric Power Development," Proc. Intern. Congress on Soil Mech., Harvard Univ., 1936, p. 286.

filter will prevent the blanket from washing through. The blanket may require riprapping for protection from high velocity in the pond or wave action if the pond is drawn down.

At the Svirstroy Dam, previously mentioned, in order to obviate the possibility of leakage at the junction of the upstream apron and the dam, due to possible cracks caused by unequal settlement, the junction was composed of flexible asphalt concrete on top of which was placed a layer of pure asphalt covered by earth fill. The junctions between the apron and the cutoff walls as well as the contraction joints in the apron were similarly treated.

In the dam shown in Fig. 15, a filter drain and blanket would have been advisable above the junction of the upstream apron and the dam, as indicated in Fig. 13, had the designers feared the possibility of a crack in the apron occurring at that place.

21. Cutoffs for Dams on Earth. Art. 19 discusses the general arrangement of cutoffs. Obviously, wherever practicable, the cutoff should be carried to impervious materials. The cutoff may consist of a concrete diaphragm, interlocking-steel sheet piling or tongue and grooved-wood sheet piling.

The only condition in which a concrete cutoff should be used is where impervious materials can be reached and where boulders prevent the use of sheet piling. Where sheeting is used, the sides of a concrete cutoff trench are disturbed and may allow ready seepage. This reduces the efficiency of such a cutoff unless carried to impervious material. Also, an upstream apron of sufficient length to be as effective as a concrete cutoff going only part way to impervious material usually proves more economical. The width of the concrete cutoff may be as small as excavation will permit. It needs no reinforcement except if tied into the dam to assist in prevention of sliding.

Steel sheet piling has been gaining favor recently for dams on earth. Where the driving is easy, the depth is not great, and particularly where jetting is feasible, light shallow arch piles may be used. At the Fort Peck Dam piles of this type of 23 and 28 lb were driven 150 ft with the aid of jets.[9] The lighter type was used under similar conditions and equal depth at the Kingsley Dam in Nebraska. However, where driving is extremely hard, heavier deep channel sections must be used.

Where boulders are present in the soil, it is sometimes extremely difficult to drive even the heaviest sections without curling the ends of the piling. Because of the liability of piles to curl, extreme care should be taken with the driving. Light driving will be more effective in cracking, moving, or otherwise passing a boulder than heavy driving. Where doubt exists, piles should be pulled at intervals for examination. Cased borings, provided with alignment devices, have been used to determine if the piles are out of line.

Wood sheet piling can be used only for shallow depths, under the most favorable conditions, and by experienced men.

Chemical grouting is the only method now known for impregnating earth to

[9] "Deep Sheet Pile Cutoff Wall for Fort Peck Dam," *Eng. News-Record*, January 10 1935, p. 35.

decrease its permeability. It is at present very expensive and is not used except for very special conditions. (See Ref. 19, Art. 40.)

22. Drains for Dams on Earth. Drains are used to carry away harmlessly any seepage which passes under the dam. The use of layers of rock, broken stone, or holes through aprons are inadequate without the protection of a filter to prevent piping of foundation material through them.

Downstream aprons of concrete are a necessary adjunct for spillway dams on earth. If the main drain shown in Fig. 13 were moved farther downstream, the upstream apron could be shortened, at the same time keeping the length of path of percolation the same. However, this procedure would increase the

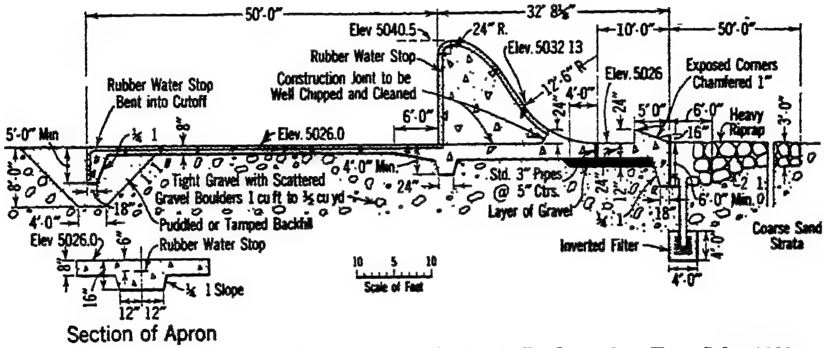


FIG. 16. Cross Cut Dam, Idaho (J. R. Sutherland, Reclamation Era, July 1938, p. 131).

amount of uplift on the dam and apron, resulting in the necessity for more concrete. Thus the location of the main drain is a matter of economy.

Model tests will indicate the effect of changing the location and depth of the main drain. The deeper it is, the more effective. If there is a relatively impervious stratum at the surface, the main drain should pierce it. For this purpose, deep sand-filled drainage wells were used at the Svirstroy Dam (Fig. 14), since the foundation was composed of horizontal layers of clay with interbedded sandy seams of greater permeability and the whole was subject to severe artesian pressure.

At the Cross Cut Dam (Fig. 16) a water-bearing sand stratum was encountered under the compact gravel, about 10 ft below the base of the dam. To relieve any possible uplift pressure in this area, a filter drain, as shown in the figure, together with 12 well points 14 ft long at other places were installed.

A filter drain may be made of dumped rock or boulders of fairly uniform size in order to provide a great percentage of large voids for the passage of water, or it may consist of perforated or open-joint pipe surrounded by crushed rock or very coarse gravel.

The exits of drains are carried out through pipes in the concrete to the face of the dam or to the top surface of the apron. In the Imperial Dam (Fig. 4) the exit pipes intersect the surface of the spillway bucket at an acute angle, so

that the suction or ejector effect of the flowing water during floods will reduce the pressure within the drain, increase its effectiveness, and reduce uplift. Care must be taken that exit pipes will not become clogged by sediment or by the nests of animals.

An ideal filter, surrounding the drain, consists of several layers of screened gravel and sand, each layer being composed of smaller grain sizes than the next one closer to the drain. The gradation of sizes must be such that the coarsest will not be washed into the rock of the drain and those of each size and the foundation will not be washed into the next layer.

Tests have indicated that particles of a given size cannot be washed through the voids of a layer composed of particles up to ten times that size.

It is, of course, impracticable to provide layers composed of grains all of one size. Frequently the gravel provided for the filter is separated into two to four sizes. In the Imperial Dam (Fig. 4) four sizes were used. Other dams have only two sizes and, for drains on a coarse gravel foundation, the entire filter might be omitted.

As it is impossible to provide a perfect filter, it is expected that, when flow first starts, a small amount of filter material may be washed into the drain. However, the flow automatically corrects any defects, provided there is sufficient material available in the filter or the foundation to provide a perfect filter after adjustment or "priming" has taken place.

For this reason, the voids in the drain proper must be sufficiently large to accommodate the material which is washed into it during the priming stage. Tests in the laboratory are easily made to simulate exactly the conditions proposed for the dam, in order to indicate the required nature of the filter and the amount which will be washed into the filter during priming.

Drains under the apron may not be needed if the main drain is sufficient to eliminate or effectively reduce uplift under the apron and to remove danger of piping. They were not used at the Imperial Dam (Fig. 4). Possibly the desired thickness of apron to withstand impact of the falling water was sufficient to balance any residual uplift.

23. Downstream Apron. The purpose of the downstream apron has been explained previously, as well as the necessity for making it heavy enough to balance all uplift pressures. The shape and details of its top surface are frequently governed by the necessity for killing the velocity of the spilling water, as explained in Arts. 26 to 39.

24. Downstream Cutoff. The downstream cutoff (see Fig. 4) is used to protect the foundation under the apron in case of retrogression of the stream bed through failure of the riprap. As explained before, it also reduces the danger of piping where drainage is not controlled.

When drains upstream from the cutoff are not used, it increases the length of path of percolation and hence increases the uplift. However, the effect on uplift of downstream cutoffs is insignificant if such drains are used.

In an attempt to remedy this condition, holes have sometimes been placed at intervals at the top of the piling to prevent its acting as a water stop. How-

ever, such holes are of no value and are a source of danger unless there is a filter drain upstream from them to prevent material from washing through.

25. Piezometers. Piezometers should be installed at several cross-sections of the dam to indicate the uplift pressures after the pond is filled. This will indicate whether or not the assumptions used in the design are correct.

A piezometer might consist of a 2-in. pipe, embedded in a small filter drain at its lower end and arranged so that the elevation of water surface within it can be easily observed, or measured with a pressure gage.

IV. CONTROL OF EROSION BELOW SPILLWAYS

By Harold A. Thomas [10]

26. Causes of Erosion. The water discharged over the spillway of a dam usually falls with but slight resistance through a height corresponding approximately to the difference of headwater and tailwater elevations, and thus acquires a velocity which is generally much higher than the natural stream velocity at the given site, thus upsetting the equilibrium of the stream by the concentration of excessively high velocities and abnormal underground pressure gradients and presenting a possible opportunity for serious erosion.

Deep erosion below dams can occur not only where the stream bed is composed of soft, granular, or pliable materials, but also where the bed is of solid rock. In the violently turbulent region below the spillway of a dam, those high-velocity eddies or flow filaments which descend to the stream bed have the property of searching out and penetrating open cracks or void spaces in the material. The resulting pressure in these crevices may be as great as that corresponding to the velocity head of the water particles—a pressure only slightly less than that due to the full head on the dam. Below dams with small head this action easily removes silt, sand, gravel, hardpan, and boulders. As the head becomes greater, ledge rock with open joints or bedding planes is attacked and lifted out in blocks or masses. If a hydraulic jump is present, the eroding potentiality of these abnormally high underground pressures is accentuated under the shallow-water region just upstream from the jump.

For dams of moderate height, say from 30 to 100 ft, experience with the erodibility of solid rock formations is extremely variable, some formations showing a surprisingly high degree of resistance and others the reverse of this. The criterion for the erosion-resisting ability of rock is obviously not the hardness, strength, toughness or nonabrasiveness of the material itself, but rather some function of the properties of the system of cleavage or bedding planes traversing this material.

The forces tending to excavate masses of solid rock below the spillway of a dam increase rapidly with the head. Where the rock surface is rough or jagged—either in its native state or because of construction operations or erosion—and the velocity of water masses striking this surface is sufficiently high, vapor

[10] Professor of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa.

spaces or cavitation pockets open up in the fluid behind the projections and the cutting and penetrating tendency is enormously accentuated. Few native rocks are so smooth and free from cleavage and bedding planes that they can be considered absolutely safe against erosion under conditions existing below a very high dam where no special provision is made for dissipating the energy of the water before it reaches the stream bed.

27. Spillway Model Tests. In the problem of designing the erosion control features of a dam, the engineer is confronted with numerous variables in addition to those pertaining to the nature of the stream bed. Among these may be mentioned the frequency and intensity of flood flows, the degree of protection to be provided for very infrequent floods, and the elevation of tailwater at various discharges.

Because of the interrelation of these variables it is difficult to standardize spillway apron designs or to be assured of satisfactory results by simply copying some existing structure. This is particularly true because certain energy-dissipating devices are especially sensitive to tailwater elevations, slight differences in tailwater depth being sufficient to change the type of performance from excellent to wretched. For these reasons, if the structure is an important one and there exists any possibility that erosion below the spillway might cause undesirable conditions, it is best to have the design checked by a model test, even though previous test results on models of apparently similar structures are available. Emphasis should be placed on the fact that experience and adequate facilities are necessary for accurate results.

Extensive modern experience with spillway models proves that these give a satisfactory reproduction of prototype conditions, insofar as the general distribution of velocities in the channel below the structure is concerned. In such models, gravel is usually used to simulate the material of the river bed. Although it is obviously impracticable to correlate the depth of the gravel scour precisely with the probable depth of scour in the prototype material, nevertheless the use of the gravel is helpful in bringing out the relative efficacy of various energy-dissipating or energy-controlling devices. Ordinary spillway models do not reproduce surface tension effects such as air entrainment and spray formation, and they do not reproduce atmospheric pressure effects such as cavitation. They do not correctly reproduce hydraulic friction effects dependent on the viscosity of the liquid, but in typical spillway models the friction loss is relatively small and is controllable by roughness adjustment on the model surfaces.

Experiments on spillway models invariably prove that excellent energy dissipation can be obtained in the models by interposing baffle piers, sills, or other obstacles in the path of the main jet. Such designs are entirely practicable for low dams. However, if the dam is of considerable height, the destructive effects of cavitation, as explained later, may prohibit the use of solid obstacles for breaking up the jet, even though such obstacles appear quite effective in the models. Difficulty may be experienced in making the latter point clear to all persons connected with the work.

28. The Hydraulic Jump. Since the theory of the hydraulic jump is associated intimately with erosion phenomena and control, it will be discussed first.

In the absence of erosion of solid materials it is obvious that energy dissipation can occur only by means of the turbulence produced by the impact of water against water, since no energy can be transmitted to a stationary solid surface by pressure against it. To obtain the dissipation of large amounts of energy within a limited space requires the production of violent turbulence. Among the various methods available for throwing a large mass of water into extremely violent turbulence and dissipating its energy, one of the most simple and effective is the use of the hydraulic jump.

For a dam whose spillway bucket is terminated by a horizontal apron or horizontal channel bed bounded by vertical side walls, the conditions pertaining to the formation of a hydraulic jump may be explained in connection with Fig. 17. As mentioned in numerous textbooks on hydraulics, a hydraulic jump can occur in a level channel of rectangular cross-section if the velocity, v, exceeds the "critical velocity" whose value is $\sqrt{gd}$. The depth, d', of water below the jump, necessary for the jump to occur, is given by

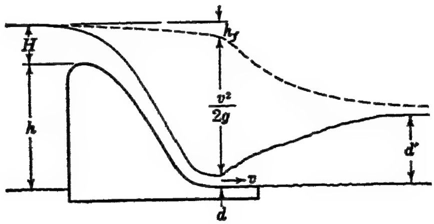


FIG. 17. The hydraulic jump below a spillway.

$$d' = \sqrt{\frac{2q^2}{gd} + \frac{d^2}{4}} - \frac{d}{2} \qquad [6]$$

where q = the discharge in cu ft per sec per linear ft of channel width, d = the depth upstream from the jump, in feet, and g = acceleration of gravity in feet per sec per sec = 32.2.

Because of friction on the spillway face the jet thickness is slightly greater than the distance between the upper and lower nappes of the corresponding free-falling jet. The latter distance may be taken from Figs. 2 and 3 of Chapter 11, for comparatively low dams. This will give the value of d with accuracy sufficient for most purposes. Where a more precise value of d is required for high dams, its evaluation can be made in theory by the usual methods of determining surface profiles for steady nonuniform flow in open channels, but two difficulties lend some uncertainty to the result: (1) information is meager regarding the accuracy of the Manning and similar formulas in extremely steep chutes, and (2) air entrainment may produce an effect similar to that of increased roughness.

However, the use of Manning's formula is the best approach which can be made at the present time and, as an approximation, the head lost in friction on the face of an ogee spillway of general proportions equivalent to that shown in Fig. 17 may be taken as equal to that in a channel of uniform depth whose length is equal to h. Using the Manning formula $V = \dfrac{1.486}{n} r^{2/3} s^{1/2}$ this loss then becomes

$$h_f = \frac{n^2 v^2 h}{2.21 d^{4/3}} = \frac{n^2 q^2 h}{2.21 d^{10/3}}$$

The value of d may then be obtained by solving the following equation by trial.

$$H + h = d + \frac{q^2}{2gd^2} + \frac{n^2 q^2 h}{2.21 d^{10/3}} \qquad [7]$$

Low values of coefficient of roughness are conservative in energy-dissipation studies, and a value of $n = 0.013$ is recommended.

As a general principle applicable to dams of all heights, it may be stated that measures for erosion control are comparatively easy of attainment and inexpensive if the natural tailwater depth at the dam site equals or exceeds the depth required to produce a jump, while their difficulty and expense increase as the tailwater depth becomes materially less than this.

In the ordinary hydraulic jump the energy of the jet is destroyed by impingement against water, a definite depth of tailwater being required to accomplish this, as indicated in Eq. 6. The horizontal distance required for reduction of the bottom velocity to approximately the mean tailwater velocity is about five times the depth of tailwater. Measures for increasing the tailwater depth to the required value and for protecting the river bed over the extensive high-velocity area under the jump are often very costly.

29. General Requirements of Erosion Control. Fig. 18 shows, diagrammatically, various types of erosion-control structures which are now used.

Various opinions are prevalent regarding the degree of erosion protection required below a dam. For dams founded on rock, the conventional method of design consists in laying off the spillway profile on an ogee curve which provides a bucket to turn the water so that as it leaves the structure it is directed downstream in a horizontal or nearly horizontal direction, as in Fig. 18*b*. In some designs the downstream edge of the bucket is at river-bed elevation, and in others it is somewhat higher. Frequently the tangent to the bucket profile at its downstream extremity is made horizontal, and sometimes a horizontal paved apron is extended some distance downstream from this point, as in Fig. 18*d*. In other designs the tangent to the downstream edge of the bucket is directed a few degrees above the horizontal, so as to give the water a slight upward component as it leaves the structure, as in Fig. 18*c* and 18*g*. In comparatively low dams on solid rock the bucket is sometimes omitted, as in Fig. 18*a*.

The formation of a hydraulic jump at the immediate toe of the spillway bucket requires that the actual tailwater depth be exactly equal to the depth d' computed by Eq. 6. If the tailwater depth is greater than this, the head of the jump will move upstream until it reaches the bucket and will then ascend the sloping face of the jet until the jump is completely submerged by the tailwater, as in Fig. 18*e*. Under this condition the turbulence of the water overlying the jet is less violent than when the jump occurs in the level channel, and therefore the rate of energy dissipation is slower and high velocities extend much farther downstream along the river bed.

On the other hand, if the actual tailwater depth is less than the computed depth d', the jump will move downstream to a location where channel friction

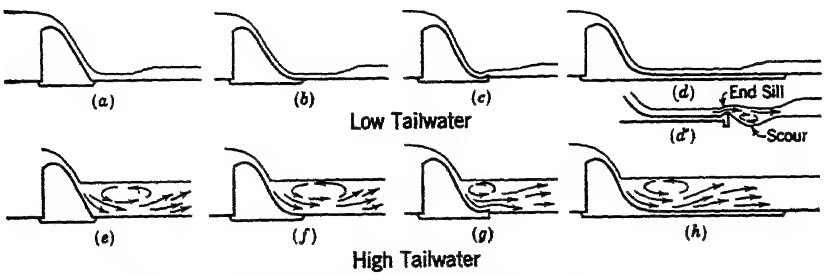


FIG. 18. Spillways with slight provision for erosion control.

has decreased the velocity of the jet and increased its depth to values consistent with a jump to the given tailwater elevation, as in Fig. 18*d*. Under the latter condition, if the jump is forced entirely off the apron the river bed will obviously be exposed to the full velocity of the main jet.

In such dams, as well as in all dams of any considerable height, modern engineering practice recognizes the necessity of obtaining more adequate erosion control than that provided by the conventional design.

In order to obtain a high degree of protection against erosion of the stream bed below the spillway of a dam, the designing engineer must provide one of the three following methods of protection or a combination thereof.

1. Means of dissipating most of the excess kinetic energy of the water before discharging it over the channel bed. This may be accomplished by the use of the hydraulic jump, as shown in Fig. 19*a*, 19*b*, and 19*f*, supplemented in some dams by baffles. When the tailwater is insufficient to create a jump near the toe of the dam, the required depth can be obtained by the creation of a "stilling pool" obtained by the construction of an auxiliary small dam, below the apron, as shown in Fig. 19*a*, or by excavating the river bed, as shown in Fig. 19*b* and 19*c*. Either of these measures is expensive. The second is especially undesirable where the rock is so stratified that the additional excavation reduces the safety factor of the structure against sliding.

When tailwater depth is more than sufficient to create a hydraulic jump, resulting in high bottom velocities, as previously explained, the depth of tail-

water can be reduced by providing a sloping apron, as shown in Fig. 19*f*, or baffles can be used to dissipate the energy, as in Fig. 19*e*.

2. Guiding the high velocity filaments along the tailwater surface in such a way that they do not penetrate to the bottom. This may be done by the aid of an end sill (Fig. 19*e*) or by an upturned bucket (Fig. 19*g*).

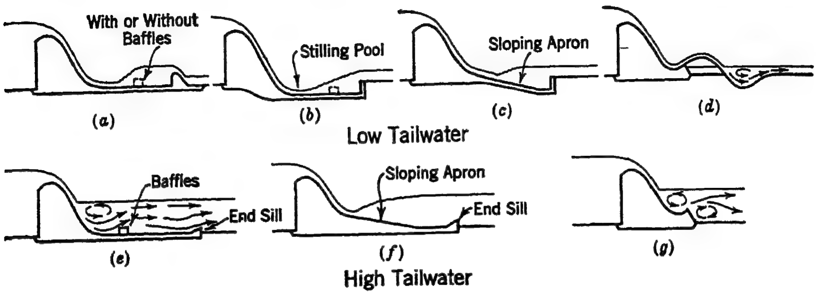


Fig. 19. Spillways with special provision for erosion control.

3. If some degree of erosion is permissible, it may be shifted from the immediate toe of the dam to a point downstream by directing the trajectory slightly upward as it leaves the bucket (Fig. 19*d*) or by the use of an end sill (Fig. 19*e* and 19*f*).

30. Baffle Piers. If baffle piers or other solid objects are anchored on the apron in the path of the jet, as in the design shown in Figs. 19*e* and 20, these assist in breaking up the jet into a mass of turbulent water, so that both the

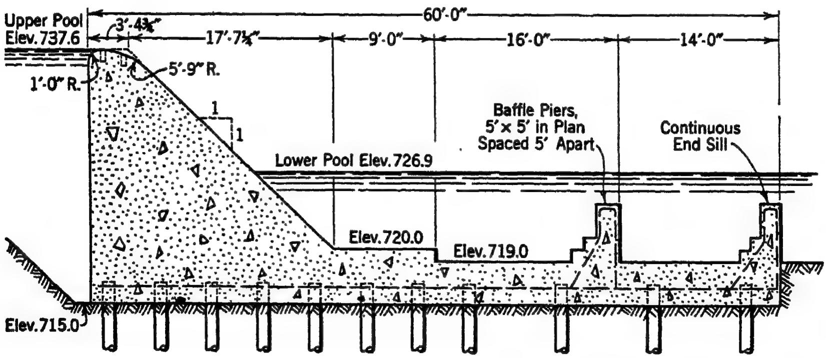


Fig. 20. Spillway and apron of dam No. 4, Monongahela River.

tailwater depth and the horizontal distance required for effective stilling are much less than in the ordinary hydraulic jump. When baffle piers are used, the tailwater depth need be only sufficient to submerge the piers enough to prevent the splash from them from being thrown high in the air, unless cavitation conditions require a greater submergence. The latter topic is discussed subsequently.

In many recently designed navigation dams on American rivers the erosion-control system is somewhat similar to that shown in Fig. 4 but with one or more rows of baffle piers on the level apron. These are very effective in aiding the dissipation of the energy of the jet and are permanent where the head is not sufficiently great to induce cavitation and where cutting materials such as sand or gravel do not pass over the spillway. Dam No. 4 on the Monongahela River (Fig. 20) illustrates the use of a single row of baffle piers and a plain or nondentated end sill. Many baffle-pier shapes have been evolved in accordance with the fancy of various designers, but some of these are too flimsy to withstand the impact of ice or logs.

Maximum energy dissipation per unit projected area of baffle pier normal to the velocity is obtained by the use of a plane vertical upstream face, while maximum structural economy, as a buttress, requires a gradually sloping downstream face. The effectiveness of a baffle pier decreases greatly if its upstream face is sloped sufficiently to permit the blows of ice or logs to glance off, and becomes practically zero if the slope of the upstream face is made flat enough merely to deflect the water without throwing it into turbulence. Baffles behave best when deeply submerged. If the submergence is too shallow it is possible for the jet to be deflected high in the air when it strikes the baffles, the action being suggestive of that in a geyser.

The best location and size of baffles can be determined only by model tests.

For very high dams, it is unfortunate that cavitation prevents the utilization of baffle piers or other angular objects inserted directly in the path of the main jet. Therefore, for high dams, only two methods of erosion control are available if the natural tailwater depth is too low for the formation of a jump: (1) building an auxiliary dam below the main structure (Fig. 19*a*) and (2) excavating a basin in the channel bed at the foot of the spillway (Fig. 19*b*).

31. Sloping Aprons. A number of modern high dams, having high tailwater, among which may be mentioned the Madden, Norris, and Shasta Dams, are provided with long sloping spillway aprons whose object is to prevent the submergence of the hydraulic jump at all discharges. Such an apron is shown in Figs. 21 and 19*f*. Experiments on models prove that a hydraulic jump showing strong turbulence in its surface roller and correspondingly excellent energy-dissipating characteristics will form in channels sloping downstream with an inclination not steeper than 1 vertical to 4 horizontal. On slopes steeper than this the jump tends to take on the submerged form, with milder turbulence in the overlying water, correspondingly slow dissipation of energy from the main jet, and greater bottom velocities. Consequently a slope of 1 to 4 or flatter is considered suitable for the sloping apron. As a result of studies on models of the spillway of the Madden Dam (Trans. Am. Soc. Civil Engrs., 1938), Mr. R. R. Randolph states that a good jump formation will obtain with the point of beginning on the curved surface of the bucket, but not if it is forced up on the steep slope of the downstream face of the dam. In order to utilize the full length of the apron, the jump should begin as far back as possi-

ble, and for the maximum expected flow it should begin at the point where the spillway bucket curves away from the face of the dam.

Where the tailwater depth is greater than that required to produce a jump, the sloping apron must obviously be high and involve a large yardage of concrete, although a portion of this may be regarded as contributing to the stability of the dam. Where the tailwater depth is less than that required to produce a jump, the downstream end of the sloping apron may terminate in a

FIG. 21. Model test of stilling basin for Loyalhanna Dam, showing stilling action during severe flood.

pool excavated in the river bed to a sufficient depth to give the required tailwater height, d', as in Fig. 19c. This design possesses the excellent feature of keeping the place of excavation some distance downstream from the toe of the main dam. The spillway apron and stilling pool of the Loyalhanna Dam is shown in Fig. 21 as an illustration of the application of the foregoing principles. The steps on the sloping spillway apron were found to be of material assistance in dissipating the energy of the main jet, as they tended to throw the latter into a sinuous form from which eddies were more readily fed off into the overlying roller of turbulent water. On a dam of greater height the use of steps on the apron and end sill might be considered inadvisable because of possible cavitation troubles.

If a sloping apron does not terminate in a stilling basin, it should usually be provided with an end sill or other device, such as indicated in Figs. 19*f*, 20, and 22, to deflect the bottom filaments of water away from the river bed. It is

possible for a sloping apron without such a device to give highly unsatisfactory results.

In general, sloping aprons are most suitable for use where the height of the dam is too great to permit the safe use of baffle piers for breaking up the main

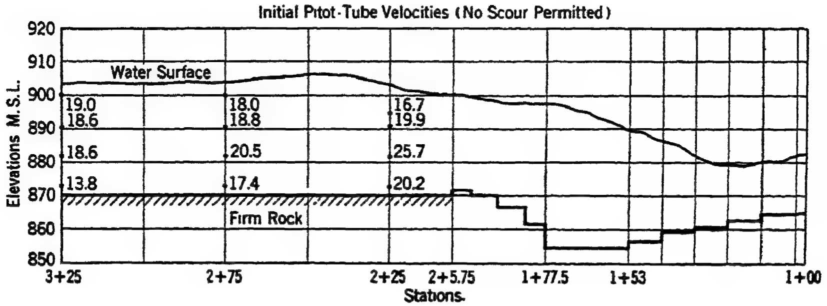


FIG. 21*a*. Same model as in Fig. 21, showing tailwater velocities (ft per sec) during an extreme flood.

jet. However, if baffle piers are employed, it is desirable to submerge these as deeply as possible in order to obtain pressure head to minimize the tendency for cavitation pockets to open up. The same remark applies to sills, steps, or other objects placed on the apron with the object of helping to break up the jet by projecting into its path. This result is accomplished most economically by entirely omitting any sloping apron, the degree of turbulence obtainable by use of the deeply submerged baffles being fully equal to that in the hydraulic jump. The undesirable effect of placing baffle piers on a high sloping apron below a dam of considerable height is illustrated in the Gatun Dam of the Panama Canal Zone. It is reported that the maintenance of the baffle piers on this structure has been a recurring source of trouble and expense.

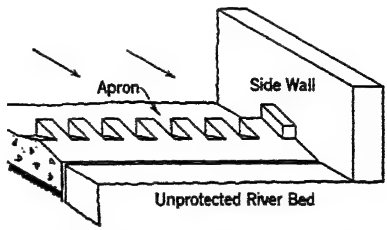


FIG. 22. The Rehbok dentated sill.

32. Upturned Bucket. Where tailwater depths are more than adequate to produce a hydraulic jump, extremely satisfactory stilling below a high dam can be obtained by the use of a strongly upturned and deeply submerged bucket, that used in the Grand Coulee Dam being shown in Figs. 19*g* and 23. This involves far less yardage of concrete than the sloping apron referred to in the three foregoing paragraphs. This design should not be used without being subjected to an exhaustive model study, as it involves a feature which might be extremely dangerous. If the tailwater gets too low the jet can push the tailwater away and can rise in the air nearly to the elevation of the headwater. Such action would be capable of destroying a powerhouse or other property located in the vicinity. Even if the tailwater is adequate for safe operation

during floods, the possibility of large quantities of water being thrown high in the air owing to the failure or accidental opening of one or more of the head gates should receive consideration.

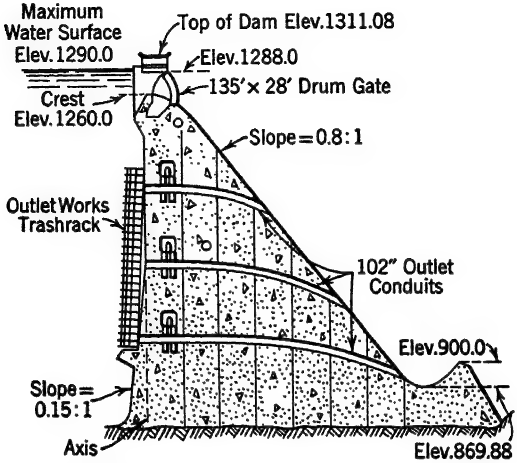


FIG. 23. Spillway of the Grand Coulee Dam.

33. Arch Dams. Arch dams are frequently designed with a vertical, or nearly vertical, downstream face. If the spillway is located on the crest of such a dam, the jet leaves the face of the structure and falls freely in the open air. If an artificial stilling pool is not provided, the jet will probably excavate a deep pool of its own, resembling the pool at the base of a natural waterfall.

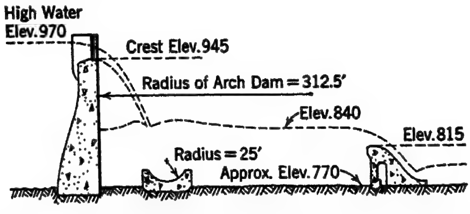


FIG. 24. Stilling basin of the Calderwood Arch Dam.

For complete stilling of a vertically falling jet a great depth of water is required. The stilling pool at the Calderwood Dam was created by the construction of an auxiliary dam a short distance downstream from the main structure. As shown in Fig. 24, a curved concrete bucket was installed at the bottom of the pool in line with the trajectory of the jet, its function being to receive the impact of those filaments penetrating to the bottom and to turn them up into the overlying water. The bucket is of a width to accommodate the trajectory of the jet at all discharges. Model experiments indicated that the jet lost little of its energy prior to striking the bucket, but the subsequent energy destruction

was complete. This stilling system at the Calderwood Dam has functioned successfully during several severe floods.

34. Low Dams. For low dams on ledge rock the problem of erosion control can usually be solved by comparatively simple means. However, for low dams founded on soft or loose granular materials—especially if planned to pass extremely large floods—the problem may present serious difficulties. These are increased by the fact that at low heads the hydraulic jump no longer has the "direct" form characterized by rapid energy dissipation, but acquires the "undular" form involving a train of standing waves extending some distance downstream. Under this wave train relatively high bottom velocities persist much farther downstream than in the case of the "direct" jump.[11] Moreover, at low heads the spillway jet corresponding to a given discharge is comparatively thick and therefore difficult to break up by baffle piers or sills unless these are made very high. However, because of the absence of cavitation at low jet velocities, the use of baffles and sills becomes permissible. Assistance in solving the erosion problem below dams on rivers with beds of soft materials is also obtained from the fact that such rivers are characterized by low velocities and correspondingly high flood stages, the latter tending to provide the deep tailwater requisite for effective jet stilling. On such rivers the tailwater during floods often submerges the spillway crest. A complicating feature in the erosion problem at many low dams arises in the use of flood gates.

In river channels of soft or loose granular materials, such a delicate equilibrium often exists between the scouring tendency of the water during floods and the resisting ability of the material that serious erosion will result from the slightest increase in velocity above that natural to the tailwater at the site. In structures on such materials, it is necessary to dissipate all the excess energy of the jet before the water passes off the paved apron. If high-velocity filaments extend into any portion of the stream downstream from the apron, it is difficult to prevent swiftly whirling eddies from reaching the bed of the stream and eroding it. Fortunately, in dams on permeable materials the requirement of a flat percolation gradient makes the total base width of the structure—dam and apron—sufficiently great to supply space for energy dissipation over the apron. In such dams, modern practice usually provides an ogee spillway followed by a wide level apron, with or without baffle piers and terminated in an end sill (see Fig. 22). The apron is usually at river-bed elevation or somewhat above, but in case the hydraulic jump is pushed off the apron at some discharge, the apron may have to be lowered below the natural channel bed. In some cases the downstream face of the spillway is sloped gradually and provided with one or more steps, either plain or dentated, to aid in spreading and breaking up the jet as it enters the jump. Fig. 4 showing a section of the Imperial Dam on the Colorado River illustrates some of the foregoing features. A single dentated step is used at the base of the spillway, and a dentated sill at the downstream edge of the concrete apron.

[11] See *Hydraulics of Open Channels*, by Boris Bakhmeteff, McGraw-Hill Book Co., New York, 1932.

35. End Sills. Where a high degree of erosion control is required, an end sill of some kind is usually placed on the downstream edge of the level apron in order to deflect residual high velocity filaments away from the river bed in the immediate vicinity and to diffuse them through the tailwater. It is not feasible to obtain effective stilling by inserting such sills directly in the path of the main jet before breaking up the greater part of its energy by means of the hydraulic jump, either with or without the aid of baffle piers.

If no baffle piers are used on the apron, best results are obtained with a dentated sill, the design patented by Dr. T. Rehbok being especially effective. This is shown in Fig. 22. Dentated sills of other designs function reasonably well. If baffle piers are used on the apron, dentates in the end sill are not required, but a plain sill with upstream face either vertical, sloping, or stepped may be used (Fig. 20). In the absence of baffle piers a plain end sill tends to produce a "bed roller" or rotating mass of water underlying the main jet just downstream from the sill and giving a strong upstream current on the river bed just below the sill.

36. Cavitation on Baffle Piers. Damage to solid materials due to cavitation occurs in the following manner: When any solid object projects into a stream of water moving at velocity, v, the pressure on the downstream side of the object is depressed below the general velocity of the stream by an amount proportional to $v^2/2g$. If the velocity is so high that the pressure at any point behind or adjacent to the object is reduced to the vapor pressure of the water, an open space or "cavitation pocket" will appear in the liquid, cavitation will occur, and the concrete will rapidly disintegrate.

To prevent cavitation the following relation must exist:

$$p_a + S - j\frac{v^2}{2g} \geqq p_v$$

where p_a = atmospheric pressure, in feet of water;

S = the necessary submergence of the pier or sill, in feet;

j = a coefficient dependent on the shape of the pier or sill (obtainable by model experiment);

$\frac{v^2}{2g}$ = the velocity head at the pier or sill, in feet;

p_v = vapor pressure of water at the given temperature, in feet of water (see steam tables);

$j\frac{v^2}{2g}$ = the reduction of pressure below that in the surrounding water.

The foregoing equation requires that the net pressure at the obstruction, represented by the atmospheric pressure, p_a, plus the submergence, S, less the reduction in pressure $j\frac{v^2}{2g}$ due to the high velocity passing the obstruction, must be equal to or greater than the vapor pressure of water, p_v.

Therefore, the minimum submergence required to prevent cavitation is

$$S = j\frac{v^2}{2g} + p_v - p_a \tag{8}$$

For an isolated cubical baffle pier resting on a horizontal plane and with a pair of its faces normal to the direction of flow, and for baffle piers having various other shapes but having vertical upstream faces, the approximate value of j is 0.68.

Ordinarily it is not feasible to make an accurate paper analysis of the cavitation potentialities of a new design for a baffle-pier group or similar stilling

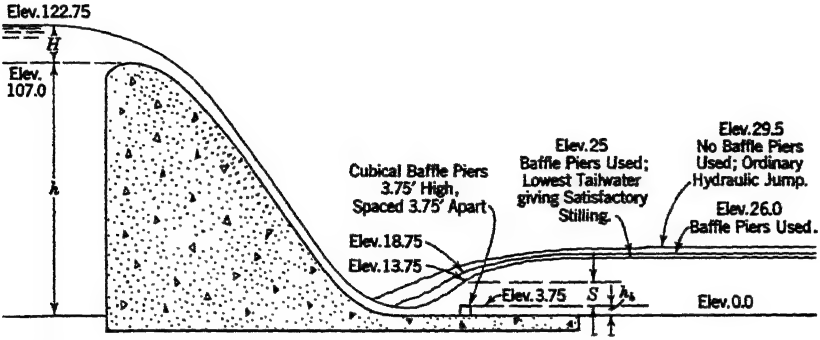


FIG. 25. Test of baffle pier effectiveness, using a spillway model built for studies on the Red Bank Dam.

device without the aid of a model test. In particular, if Eq. 8 is to be used, such a test is necessary to determine the relation between the submergence, S, of the tops of the baffle piers and the tailwater depth, d', to determine the value of j applicable to the given pier shape and grouping, and to determine the actual velocity, v, with which the water strikes the baffle piers. In an ordinary open-air model the pressures obtaining on the top, lateral, and downstream faces of the baffle piers may be determined directly by providing piezometer openings in those faces. If any one of the resulting pressures, when transferred to the prototype scale, is below the vapor pressure of the water, it is evident that cavitation will occur in the prototype. In the absence of such piezometer openings, and where the shape of the baffle pier is such that j is known, the cavitation potentiality may be determined approximately by computation, as in the following example, which is based on a test of a model of the spillway shown in Fig. 25, all quantities in the figure being referred to the prototype scale. In this model test, when no baffle piers were used, the minimum tailwater depth requisite to prevent the hydraulic jump from being pushed off the apron was 29.5 feet. Using a single row of cubical baffle piers 3.75 feet on a side and spaced the same distance apart, the tailwater depth could be reduced to about 25 feet while maintaining a stable jump, but further reduction permitted the spurting up of geysers of water at the baffle piers, the

submergence of the tops of the latter being approximately 10 feet. With the same baffle piers a steadier jump was obtained at a tailwater depth of 26 feet, the submergence of the tops of the piers under this condition being 15 feet, and the approximate mean velocity head of the water approaching the piers (as determined by a Pitot tube in the model) being 91.0 feet. Taking the atmospheric pressure and the water-vapor pressure as equivalent to 34 feet and 1 foot respectively, and using $j = 0.68$, the submergence required to prevent cavitation, according to Eq. 8, would be $S = 0.68(91) + 1 - 34 = 29$ feet. Since this is much greater than the actual submergence of 15 feet, serious cavitation would occur at the baffle piers in the prototype. Computations such as the foregoing should be considered as giving approximate rather than precise results, because of the difficulty of determining or estimating the actual mean velocity of the water just before it strikes the baffle piers. Pitot tube tests indicate the velocity distribution in the jet at this point to be quite nonuniform.

At Carnegie Institute of Technology special apparatus has been constructed for subjecting the entire region below a model spillway to subatmospheric pressure of absolute value conforming to the linear model scale, so that cavitation pockets of the prototype are reproduced in the model in their correct size and location, visual observation being permitted by a thick plate-glass window. Where it is desired to study or modify a baffle pier or end sill design with the object of eliminating cavitation or minimizing its harmful effects, this apparatus provides outstanding advantages over ordinary open-air installations. Use of this apparatus is developing the possibility of designing baffle piers of such shape as to cause the cavitation pockets to be surrounded entirely by flowing water, and not in contact with the concrete surface of the pier itself or the adjacent floor, thereby eliminating or greatly reducing the occurrence of pitting.

37. Jet Deflectors and Stilling Pools. It has already been mentioned that when the tailwater below a spillway is too shallow for the formation of a hydraulic jump and the construction of a stilling pool is considered undesirable, an upcurved spillway bucket may be used, so that the jet is deflected slightly upward into the air and strikes the tailwater some distance downstream from the bucket. With this arrangement erosion of the stream bed usually occurs at the point where it is intersected by the trajectory of the jet. This erosion may be eliminated in places where it is feasible to flare out the jet laterally into a thin sheet before it strikes the tailwater. Such an opportunity exists if the jet is from an outlet conduit or tunnel where the width of the issuing stream is much less than that of the tailwater channel. The thin, fan-shaped sheet of water tends to skip along the tailwater surface at the point of impact, and has only slight penetration. Fig. 26*a* shows a deflector design used for flaring out the jet from the outlet conduits of the Bluestone Dam, and Fig. 26*b* shows the jet from the model of a similar deflector proposed for the Red Bank Dam. A deflector used to spread the jet from the outlet tunnel at the proposed Youghiogheny Dam proved to be much more economical than a stilling pool of the conventional design.

Stilling pools are frequently used at the outlets of tunnels or of steep chutes, and are usually designed to permit lateral spreading of the jet before it enters the hydraulic jump. It is easier to break up a wide shallow jet than to break up a deep narrow one, whether by the plain hydraulic jump or with the aid of steps, baffles, or sills. The principles involved in the design of such pools are

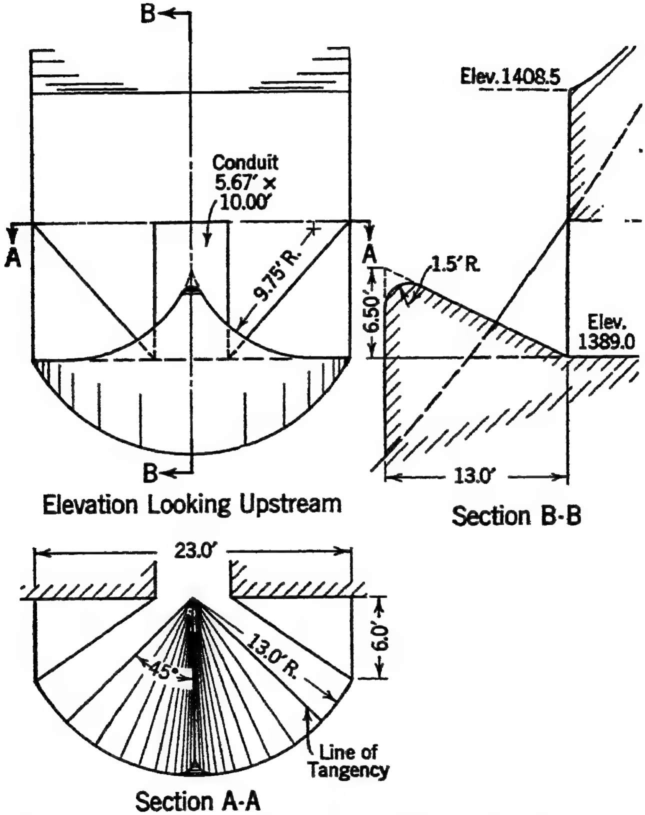


FIG. 26*a*. Jet deflector for outlet conduits of the proposed Bluestone Dam.

identical with those already described in connection with the design of ordinary stilling basins below the spillways of dams.

38. Uplift Under Apron. In a level channel the water on the upstream side of a hydraulic jump is shallower than that on the downstream side. The general shape of the jump is shown in Fig. 17, and the sloping length is about six times the rise in water surface. If no concrete apron is provided and the natural rock is stratified horizontally, pressure from the deep water downstream from the jump may find its way into the strata and, if sufficient vertical drainage is not present, may exert throughout 100 per cent of any layer an

uplift pressure equal to the depth of tailwater. Under the shallow portions of the jump this uplift would not be balanced by an equal depth of water, and the rock layers might be lifted out if the pressure is sufficient.

This uplift pressure must also be considered under concrete aprons and stilling basins which, unless drainage is provided, require sufficient weight of concrete or anchorage to the rock to make them stable.

Fig. 26*b*. Model test of jet deflector similar to that of Fig. 26*a*, but designed for the proposed Red Bank Dam.

The uplift pressure may be relieved by drain holes drilled through the concrete and into the rock sufficiently far that the weight of the submerged concrete and rock pierced by the drill hole will balance the depth of water regained by the jump. Slanting the exit of the drains downstream causes the high velocity of the water to form a partial vacuum in them which increases the factor of safety against failure due to uplift.

When the rock is not stratified horizontally and the only danger of uplift is between the concrete and the rock, the concrete may be anchored to the rock by steel bars grouted into holes drilled into the rock.

In some places a concrete or grouted cutoff, surrounding the area of low water, has been provided to reduce uplift.

When the foundation is earth and, as is always required, is protected by concrete in the form of an apron or stilling basin, exactly the same problem arises, and the same remedial measures may be taken, except that here the concrete cannot be anchored. Filter protected drains, to relieve uplift pressure

at the bottom of the concrete, may be provided, but the pore pressures in the foundation below the concrete should also be investigated as described in Art. 14.

In the foregoing discussion, the problem of uplift has been treated with neglect of possible uplift from headwater in the reservoir. Such complications can be treated by the theories previously outlined in this chapter.

39. Retrogression. In preceding articles there have been described the necessary provisions for destroying the energy of the water spilling over the dam and for reducing tailwater velocity as close as possible to that existing under natural conditions.

However, it is obviously impossible to reduce velocities leaving the protective works to less than the natural velocity for a given flow, and natural velocities frequently scour the bed of the stream. That is, under natural conditions, the beds of many streams lower during the rising stage of the flood and build up again, during the falling stage, by deposition of material carried by the flood.

Owing to the presence of the reservoir created by the dam, this material carried by the stream all deposits in the deep water above the dam and is not available to return the stream bed to normal elevation after a flood and permanent retrogression results.

For these reasons, the bed of the stream adjacent to the downstream end of the protective works must be treated so that, when retrogression occurs, the foundation for the lower end of the protective works will not be affected.

A usual procedure is to provide a cutoff of concrete or piling at the toe of the protective works, supplemented by riprap of large stones (Fig. 4).

The riprap below the apron is intended to settle and pave a new slope from the toe of the apron to the new level of the stream, and the sheet piling is intended to protect the apron from undermining in case the riprap directly at the toe settles some.

In case the toe of a spillway dam is higher at the ends than at the middle, the water passing over each end, if not taken care of, will flow parallel to the dam toward the main channel. For steep slopes, this flow may acquire velocities sufficiently great to scour the foundation at the toe of the dam or the end of the apron. This condition should be avoided by the construction of stone fill, masonry, or other suitable training dikes at intervals extending from the dam to a point downstream far enough from the dam to obviate the possibility of damage to that structure. Sometimes the training dikes are supplemented by a system of canals, parallel to the river, in order to provide a gradual descent.[12] Such construction is obviously expensive, particularly for large flows, and often necessitates the limitation of the length of spillway to the width of the level portion of the river bed. A small auxiliary dam at the downstream end of that part of the apron which is on side slopes, to confine the flow to the apron and lead it to the river channel, may be used.

[12] See "The Laguna Dam," *Eng. News*, Feb. 9, 1905, and Feb. 27, 1908.

V. BIBLIOGRAPHY

40. Arrangement. The following references are arranged approximately as to subjects. However, some may contain data on several subjects.

General:

1. W. J. Mead, "Engineering Geology for Dam Sites," *Trans. Second Congress on Large Dams*, Washington, D. C., Vol. 4, 1936, p. 171.
2. I. B. Crosby, "Geology for Dam Sites," presented to Power Division, Am. Soc. Civil Engrs., San Francisco meeting, July 1939.
3. Khosla, Bose, and Taylor, "Design of Weirs on Permeable Foundations," Pub. No. 12, *Control Board of Irrigation*, Simla, India, 1936.
4. Extensive bibliography contained in following Item 5.

Examples of Design:

5. Over 150 examples of design given in supplement to Item 7 by E. W. Lane, published in pamphlet form by Am. Soc. Civil Engrs. in 1939.
6. *Dams and Control Works*, 2nd Edition, U. S. Bureau of Reclamation, 1938.

Uplift, Seepage, and Line of Creep:

7. E. W. Lane, "Security from Under-Seepage, Masonry Dams on Earth Foundations," *Trans. Am. Soc. Civil Engrs.*, 1935, p. 1235.
8. L. F. Harza, "Uplift and Seepage Under Dams on Sand," *Trans. Am. Soc. Civil Engrs.*, 1935, p. 1352.
9. J. A. H. Bratz, "Pressures Due to Percolating Water and Their Influence Upon Stresses in Hydraulic Structures," *Trans. Second Congress on Large Dams*, Washington, D. C., 1936, Vol. 5, p. 43.

Flow Net:

10. "The Electrical Analogy for Percolating Waters in Connection with the Design of Earth Dams and Other Hydraulic Structures in the Los Angeles District," Tech. Memo, U. S. Engrs. Office, Los Angeles, Calif., May 1939.
11. H. M. Hill, "Seepage through Foundations and Embankments Studied by Glass Models," *Civil Eng.*, Jan. 1934, p. 32.
12. Lane, Campbell, and Price, "The Flow Net and the Electrical Analogy," *Civil Eng.*, Oct. 1934, p. 510.

See also Items 3 and 7 to 9.

Grouting:

13. J. B. Hays, "Improving Foundation Rock for Dams," *Civil Eng.*, May 1939, p. 309
14. "Thousands of Holes Grouted Under Norris Dam," *Eng. News-Record*, Nov. 21, 1935, p. 699.
15. R. McC. Beaufield, "Technique of Pressure Grouting of Foundations," *Western Construction News*, Oct. 10, 1928.
16. F. H. Kellog, "Clay Grouting at Madden Reservoir," *Eng. News-Record*, Oct. 6, 1932, p. 395.
17. Jones and Minear, "Grouting the Foundation of Boulder Dam," *Civil Eng.*, Dec. 1936, p. 810.
18. "Grouting Seals Alcova Dam," *Eng. News-Record*, Sept. 3, 1936, p. 323.
19. J. D. Lewin, "Grouting with Chemicals," *Eng. News-Record*, Aug. 17, 1939, p. 61.
20. G. W. Christians, "Asphalt Grouting Under Hales Bar Dam," *Eng. News-Record*, May 20, 1926, p. 798.

CHAPTER 4

HYDRAULIC MODEL STUDIES

By George E. Barnes [1]

1. Introduction. Hydraulic model studies are finding increasing application for checking and modifying the analytical designs of hydraulic structures. Hydraulic models are usually constructed on a scale such that the scale ratio (L_r) of prototype to model is from 10 to 100. If the models are too large, their cost is too great; if too small, tests cannot be made with the necessary precision. After the scale ratio is determined the results obtained from the model tests are multiplied by their proper quantity ratios to obtain the anticipated condition in nature. The model test provides a preview of

Fig. 1. Model.

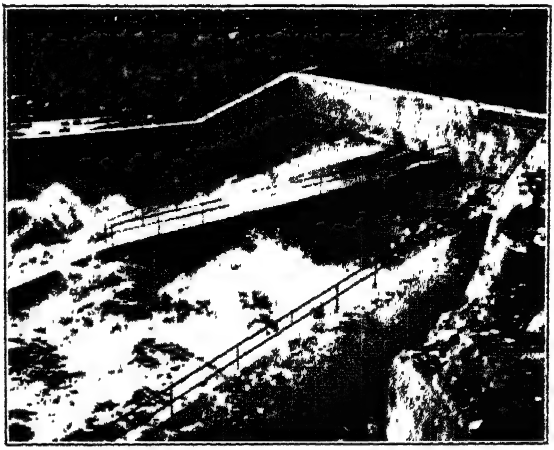


Fig. 2. Prototype.

Stilling basin for Mohawk Dam.

the behavior of the structure under operating conditions, and it gives results of both qualitative and quantitative value. In Fig. 1 is shown a view of the laboratory model of the stilling basin at the Mohawk Dam of the Muskingum Watershed Conservancy District in Ohio with a discharge equivalent to 25,000 sec ft in the prototype. In Fig. 2 is shown the prototype actually discharging 25,000 sec ft during the flood of January 28, 1937. A comparison of the two photographs gives a rough idea of the extent to which flow conditions in the prototype were predicted by the model test.

[1] Professor of Hydraulic and Sanitary Engineering and Head of the Department of Civil Engineering, Case School of Applied Science.

There are now available a number of well-equipped hydraulic laboratories, staffed with men of experience in the field of hydraulic experimentation, and as there are many pitfalls for the amateur in this field, it is believed that hydraulic model testing should be entrusted only to such laboratories.

2. Essential Considerations of Similarity. In order to apply the results of a hydraulic model test to an actual structure, it is necessary that the requirements of hydraulic similarity shall be met. In his book, *Hydraulic Laboratory Practice*, John R. Freeman gives the following definitions: "The principle of hydraulic similitude may be briefly, but incompletely, defined as that controlling the behavior of a current of water in relation to the channel through which it flows, by reason of which (within limits) this behavior will be relatively the same in a larger water course or structure as that found in a small scale model."

Two flow systems (as prototype and model) are dynamically similar when boundary conditions and stream lines are geometrically similar and when homologous forces in the two systems bear a constant ratio to each other. When dealing with moving water there may be present forces due to any or all of the following, in addition to the ever-present inertia forces: (*a*) gravity, (*b*) viscosity, (*c*) elasticity, and (*d*) surface tension. The requirements for hydraulic similitude may conveniently be expressed in algebraic form, with forces given in terms of their component elements and dimensions (length, velocity, mass, etc.). By reducing these algebraic expressions to their most convenient forms, direct criteria for similarity are obtained.

3. Criteria for Similarity. The criteria are dimensionless ratios, the terms depending on the type of forces present. Being dimensionless, the absolute values will not change with the system of units adopted in the analysis. If two systems are to be dynamically similar the ratio must have the same absolute value for each. The dimensionless ratios most used by hydraulic experimenters are the Froude number and the Reynolds number as follows:

Forces Acting	*Dimensionless Ratio*	*Name*	*Common Designation*
Gravity	$\frac{V^2}{gL}$	The Froude number	F
Viscosity	$\frac{VL\rho}{\mu}$	The Reynolds number	R

where V = velocity;

L = any length parameter influencing flow (diameter, hydraulic radius, etc.) (Note that L is usually diameter. Where other length functions such as head, length, etc., are used, the value will differ);

ρ = mass density of fluid (w/g). For water at 60° F = 62.4/32.2 = 1.94 slugs per cu ft;

μ = Coefficient of viscosity. For water at 60° F = 0.0000236 lb sec/ft^2;

ν = kinematic viscosity = μ/ρ. For water at 60° F = 0.0000236/1.94 = 0.00001217 sq ft/sec;

g = acceleration of gravity = 32.2 ft per sec per sec.

The dimensionless ratio involved in similarity as to surface tension is Weber's number and as to elasticity, Cauchy's number. However, since the effect on hydraulic flow of surface tension and elasticity is secondary or negligible, only gravity and viscosity are treated in this text. Two systems will be dynamically similar, assuming that both gravity and viscosity influence flow, only when the Froude and Reynolds numbers are each numerically the same in each system. For practical reasons it may be impossible to satisfy these conditions simultaneously, and accordingly similarity with respect to the dominant force alone becomes the governing consideration.

4. Types of Fluid Flow. Three types of flow interest the engineer. These are (1) laminar flow, as in smooth pipes at very low velocities or as seepage through granular material, etc., where R is less than, say 2500, taking L as diameter, and friction is proportional to velocity; (2) turbulent flow in short pipes or in open channels with higher velocities where R is greater than, say 2500, and friction is proportional to V^2; and (3) shooting flow with very high velocities in open channels where V is greater than $\sqrt{g \times \text{depth}}$ (the velocity at which a given quantity of water flows with minimum energy content at given or "critical" depth). In (1) viscosity forces are dominant and the Reynolds number is the criterion of dynamic similarity. In (2) and (3), viscosity forces are of negligible influence, gravity forces being dominant, and the Froude number is the criterion of similarity. As stated in Art. 3, surface tension or capillarity is neglected in this discussion. The same is true of elasticity, since for most problems water is assumed incompressible.

At the U. S. Waterways Experiment Station it was found that a good practical criterion for insuring turbulent flow in river models was to keep the velocity in feet per second multiplied by the depth in feet = 0.02 or more. It is, of course, possible for the model to have shooting flow and the condition to insure against this flow is that velocity be less than $\sqrt{g \times \text{depth}}$, thus insuring a depth greater than the critical depth.

5. Significance of the Froude Number. Since most hydraulic model studies deal with turbulent flow, the Froude number is of special interest. With due allowance for friction effects, and for the ordinary range of studies in free fall, flow over spillways, open channel flow, flow in intakes, penstocks, etc., virtual dynamic similarity obtains when the Froude number is the same for model and prototype.

Assuming that in a given case the Froude number is the criterion for similarity, an immediate consequence is to establish implicit relationships between quantities in the prototype and homologues in the model, as follows: Let V_p, Q_p, A_p, R_p, S_p, L_p, D_p, n_p, and other common symbols denote velocity, discharge, area, hydraulic radius, slope, length, diameter, roughness coefficient, etc., for the prototype and let

$$V_m,\ Q_m,\ A_m,\ R_m,\ S_m,\ L_m,\ D_m,\ n_m$$

denote corresponding terms in the model. Let L_r denote the scale ratio of

prototype to model. Then if F (the Froude number) is the same for model and prototype, the following relations are readily derived:

$$F = \frac{V_p^2}{gL_p} = \frac{V_m^2}{gL_m}, \quad V_p = V_m\left(\frac{L_p}{L_m}\right)^{1/2} = V_m L_r^{1/2}$$

and also

$$A_p = A_m(L_r)^2$$

and therefore

$$Q_p = Q_m L_r^{5/2}$$

These and other relationships derived in similar manner are tabulated below:

For dynamic similarity with respect to gravitational forces: Units of length (head, hydraulic radius, diameter, length, depth, width, etc.)

Length	$L_p = L_m(L_r)$
Time	$T_p = T_m(L_r)^{1/2}$
Velocity	$V_p = V_m(L_r)^{1/2}$
Areas	$A_p = A_m(L_r)^2$
Discharge	$Q_p = Q_m(L_r)^{5/2}$
Slopes, accelerations, constants	$C_p = C_m(L_r)^0$
Power	$P_p = P_m(L_r)^{7/2}$
Work	$E_p = E_m(L_r)^4$

6. Significance of the Reynolds Number. If the character of flow is such that the Reynolds number is the criterion for similarity (as in laminar flow or flow through granular material) other implicit relationships between corresponding quantities would be involved, dissimilar to those for the Froude number but derived in a similar manner. Probably the most frequent use which the hydraulic experimenter makes of the Reynolds number is to make sure that, in the model tests involving turbulent flow, the value of R for the model exceeds 2500 (using L as diameter).

7. Coefficient of Roughness in Model Tests. Where roughness of channel plays a large part, such as in rivers, it becomes necessary to determine a roughness coefficient for the model which will make it dynamically similar to the prototype. Roughness ratios, determined from the well-known hydraulic formulas, such as Kutter, Bazin, or Manning, are commonly used.

Using Manning's formula, $V = \dfrac{1.486R^{2/3}S^{1/2}}{n}$ and making the slopes in prototype and model equal, there follows:

$$S_p = S_m = \frac{V_p^2 n_p^2}{1.486^2 R_p^{4/3}} = \frac{V_m^2 n_m^2}{1.486^2 R_m^{4/3}}$$

Transposing and eliminating

$$\left(\frac{n_m}{n_p}\right)^2 = \left(\frac{V_p}{V_m}\right)^2\left(\frac{R_m}{R_p}\right)^{4/3} = [(L_r)^{1/2}]^2\left(\frac{1}{L_r}\right)^{4/3} = \left(\frac{1}{L_r}\right)^{1/3}$$

Hence $n_m = n_p\left(\frac{1}{L_r}\right)^{1/6}$

where n_m is coefficient of roughness in model;
n_p is coefficient of roughness in prototype;
and L_r is the scale ratio.

8. Examples in Hydraulic Similarity.

Example 1. A circular conduit, 20 ft in diameter, flows under pressure with a velocity of 10 ft per sec. What is the limiting scale ratio to insure turbulent flow in the model conduit? The Reynolds number must be greater than 2500. Using D as diameter and letting L_r = scale ratio

$$R = \frac{V_p D_p \rho}{\mu} = \frac{V_m D_m \rho}{\mu}$$

But

$$V_m = V_p\left(\frac{1}{L_r}\right)^{1/2} = 10\left(\frac{1}{L_r}\right)^{1/2}$$

and

$$D_m = D_p\left(\frac{1}{L_r}\right)$$

ρ = mass density = 1.94 slugs per cu ft;
μ = coefficient of viscosity = 0.0000236 for water at 60° F;
R = 2500, as stated above.

Substituting the numerical quantities in the above equation:

$$2500 = \frac{10\left(\frac{1}{L_r}\right)^{1/2} \times 20\left(\frac{1}{L_r}\right) \times 1.94}{0.0000236}$$

$$L_r = 351$$

Obviously such a model scale would be too small for practical observations, and the selection of any practical scale will insure turbulent flow in the model.

Example 2. Assume in Example 1 that the model is built to a scale of 20 and that the conduit in the prototype is of concrete, having a Kutter's coefficient of n = 0.015. How smooth must the conduit be for equivalent surface roughness and energy loss per foot?

Referring to the paragraph on "Coefficient of Roughness in Model Tests"

$$n_m = n_p\left(\frac{1}{L_r}\right)^{1/6} = 0.015\left(\frac{1}{20}\right)^{1/6} = \frac{0.015}{1.65} = 0.0091$$

Example 3. A river channel is 36 ft deep, with a hydraulic radius of 30 ft, flowing with average velocity of 5.0 ft per sec. If the Reynolds number is to be greater than 2500 (using D as diameter) in order to obtain turbulent flow in the model, what is the smallest scale permissible?

As in Example 1, $R = \dfrac{V_p D_p \rho}{\mu} = \dfrac{V_m D_m \rho}{\mu}$, wherein R must not be less than 2500. In this formula the value of $R = 2500$ is based on pipe diameter. If a similar formula employs the hydraulic radius, $D/4$, then the value of R must be divided by 4 and becomes 625.

$$R = \frac{V_p R_p \rho}{\mu} = \frac{V_m R_m \rho}{\mu}$$

in which R_p and R_m are the hydraulic radii of the prototype and the model. Let L_r = scale ratio. Then

$$V_m = V_p \left(\frac{1}{L_r}\right)^{1/2} = 5.0 \left(\frac{1}{L_r}\right)^{1/2}$$

and

$$R_m = R_p \left(\frac{1}{L_r}\right) = 30 \left(\frac{1}{L_r}\right)$$

ρ = mass density. For water at 60° F = 62.4/32.2 = 1.94 slugs per cu ft;
μ = coefficient of viscosity. For water at 60° F = 0.0000236;
R = 625, as stated above.

Substituting the numerical quantities:

$$625 = \frac{5.0 \left(\frac{1}{L_r}\right)^{1/2} \times 30 \left(\frac{1}{L_r}\right) \times 1.94}{0.0000236}$$

$$L_r = 730$$

Example 4. An open rectangular flume with shooting flow is to form the hydraulic jump. Width 10 ft, depth before jump 2.0 ft, velocity before jump 40 ft per sec. The hydraulic jump formula

$$D_2 = \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}} - \frac{D_1}{2}$$

in which D_1 and V_1 are depth and velocity before jump and D_2 is depth after jump, is based upon gravitational forces only. Using the Froude number for similarity, what will be the characteristics of the jump in the model on a scale of 16?

By the above formula, $D_2 = 13.1$ ft. $Q = 10 \times 2 \times 40 = 800$ sec ft.

$$F = \frac{V_1^2}{gD_1} = \frac{40^2}{32.2 \times 2.0} = 24.8$$

which also holds for the model in

$$F = \frac{\left[40\left(\frac{1}{L_r}\right)^{1/2}\right]^2}{32.2 \times 2\left(\frac{1}{L_r}\right)} = 24.8$$

where L_r is the scale ratio.

By the relationship previously stated, the model will have the following dimensions:

$$D_1 = 2.0\left(\frac{1}{L_r}\right) = \frac{2}{16} = 0.125 \text{ ft}$$

$$V_1 = 40\left(\frac{1}{L_r}\right)^{1/2} = 40\left(\frac{1}{16}\right)^{1/2} = 10 \text{ ft per sec}$$

$$Q = 800\left(\frac{1}{L_r}\right)^{5/2} = 800\left(\frac{1}{16}\right)^{5/2} = 0.781 \text{ sec ft}$$

$$D_2 = 13.1\left(\frac{1}{L_r}\right) = 13.1\left(\frac{1}{16}\right) = 0.82 \text{ ft}$$

Incidentally the above model values for D_1 and velocity, if substituted in the hydraulic jump formula, will give $D_2 = 0.82$ ft, thus verifying the similarity condition.

Example 5. A concrete dam of ogee section is 22 ft high, with base 24 ft wide, and has a crest length of 42 ft. A model is built on a scale of 10 and the following observations are made:

Height of model = 2.2 ft.
Head on model crest = 0.467 ft.
Length of model crest = 4.2 ft.
Observed discharge of model = 5.25 sec ft.
Thickness of nappe on bucket = 0.117 ft.
Hence velocity of flow on bucket $= \dfrac{5.25}{4.2 \times 0.117} = 10.68$ ft per sec.

What is the discharge, head, depth, and velocity of flow on the bucket in the prototype?

Discharge, $Q_p = Q_m(L_r)^{5/2} = 5.25(10)^{5/2} = 1660$ sec ft

Head, $H_p = H_m(L_r) = 0.467(10) = 4.67$ ft

Depth on bucket, $D_p = D_m(L_r) = 0.117(10) = 1.17$ ft

Velocity of flow on bucket, $V_p = V_m(L_r)^{1/2} = 10.68(10)^{1/2} = 33.77$ ft per sec

As a check on the above quantities:

$$\text{In prototype } Q_p = CLH^{3/2} = 3.92 \times 42 \times 4.67^{3/2} = 1660 \text{ sec ft}$$

$$= 1.17 \times 42 \times 33.77 = 1660 \text{ sec ft}$$

$$\text{In model } Q_m = CLH^{3/2} = 3.92 \times 4.2 \times 0.467^{3/2} = 5.25 \text{ sec ft}$$

$$= 0.117 \times 4.2 \times 10.68 = 5.25 \text{ sec ft}$$

9. Bibliography.

JOHN R. FREEMAN, *Hydraulic Laboratory Practice*, edited by Am. Soc. Mech. Engrs.

H. D. VOGEL, "Practical River Laboratory Hydraulics," *Trans. Am. Soc. Civil Engrs.*, Vol. 100, 1935, p. 118.

H. D. VOGEL, "Hydraulic Laboratory Results and Their Verifications in Nature," *Trans. Am. Soc. Civil Engrs.*, Vol. 101, 1936, p. 597.

GEORGE E. BARNES and J. G. JOBES, "Construction and Testing of Hydraulic Models, Muskingum Watershed Project," *Trans. Am. Soc. Civil Engrs.*, Vol. 103, 1938, p. 227.

RICHARD R. RANDOLPH, "Hydraulic Tests on the Spillway of the Madden Dam," *Trans. Am. Soc. Civil Engrs.*, Vol. 103, 1937, p. 1080.

BENJAMIN F. GROAT, "Theory of Similarity and Models," *Trans. Am. Soc. Civil Engrs.*, Vol. 96, 1932, p. 273.

B. A. BAKHMETEFF and A. E. MATZKE, "The Hydraulic Jump in Terms of Dynamic Similarity," *Trans. Am. Soc. Civil Engrs.*, Vol. 101, 1936, p. 630.

H. D. VOGEL and J. P. DEAN, "Geometric versus Hydraulic Similitude," *Civil Eng.*, Aug. 1932, p. 467.

HUNTER ROUSE, "Analytical Approach to Experimental Hydraulics," *Civil Eng.*, Nov. 1934, p. 563.

D. C. McCONAUGHY, "Boulder Dam Spillways," *Eng. News-Record*, Apr. 4, 1935, p. 480.

E. MEYER PETER and HENRY FAURE, "Boulder Dam Spillways," *Eng. News-Record*, Oct. 25, 1934, p. 520.

J. C. STEVENS, "Bonneville Scour Prevention Study by Model," *Eng. News-Record*, Jan. 14, 1937, p. 61.

M. A. HOGAN and E. F. GIBBS, "Similarity and Its Application to Problems of Fluid Flow," *Concrete and Construction Eng.*, Vol. 31, July 1936, p. 385.

A. H. GIBSON, "Use of Models in Hydraulic Engineering," *Trans. Inst. Water Engrs.*, Vol. 39, 1934, p. 172.

C. M. STANLEY, "Study of Stilling Basin Design," *Trans. Am. Soc. Civil Engrs.*, Vol. 99, 1934, p. 490.

E. BUCKINGHAM, "Model Experiments and the Forms of Empirical Equations," *Trans. Am. Soc. Mech. Engrs.*, Vol. 37, 1915, p. 263.

Am. Soc. Civil Engrs. Manuals of Engineering Practice No. 25, "Hydraulic Models," (1942).

C. V. DAVIS, *Handbook of Applied Hydraulics*, McGraw-Hill Book Co.

CHAPTER 5

FLOOD FLOWS

I. FLOOD PEAKS

1. General. The determination of the required spillway capacity to insure the safety of the dam is not an exact science. A number of methods of investigation have been suggested and used, but no one method yet devised can be said to indicate accurately the required spillway capacity. Such capacity must be based on an intelligent study of all the available data as to the flood-producing capacity of the river subjected to all possible methods of analysis, but the final determination must be a matter of judgment of an engineer fully conversant with the underlying principles of the phenomenon of floods.

It will be shown later that there is no stream in the United States which has been gaged for a sufficient number of years to provide all the data required for a complete knowledge of its flood-producing characteristics. Thus we must be guided also by what has happened on other streams.

Where there is no storage at the site to smooth out the peak of the flood, the maximum rate of discharge is of primary interest and the total volume in the flood is seldom, if ever, of importance. However, if there is a reservoir above the dam, such as a flood control basin, or if considerable storage above the spillway of storage or power dams is available so that, as described later, the flood will be smoothed out and the peak discharge considerably reduced, the total volume in the flood is of as much importance and frequently of greater importance than the peak discharge. The shape of the hydrograph is also of importance.

Although the peak discharge and the total volume are affected by much the same factors and are therefore definitely related, the methods used to estimate them are radically different. The subject, therefore, naturally divides into two parts. The subject of peak flows will be discussed in Part I of this chapter and the total volume and shape of the hydrograph will be covered in Part II.

2. Peak Flows—General. The most logical beginning of a study of probable flood peaks on a stream is an investigation of the record floods which have occurred in a representative surrounding section of the country. This will furnish the basis for the preliminary evidence. The stream in question is assumed to have flood characteristics just as great as those which have furnished record-breaking data, and this assumption will not be changed unless evidence to the contrary is forthcoming. There is, of course, the possibility

that the evidence may show even greater flood characteristics than the worst performers of record. This evidence which may be investigated consists of:

1. Comparison of the physical characteristics of the river with those of record-breaking streams (Art. 4).

2. Comparison of coefficient of variation with the coefficients of record-breaking streams (Art. 7).

3. Physical indications of past floods (Art. 8).

4. Comparison of the "lag," or time from the midpoint of a short storm to the peak of the resulting hydrograph (Art. 26).

3. Record Floods of History. For a study of record-breaking floods in any section of the country, Table 1 will be found useful. This table contains a list of unusual flood discharges in the United States and other countries, compiled from original tables by Jarvis [1] and Creager,[2] brought up to date from their files as of November 1941. The table includes only those floods which are necessary to define an enveloping curve for each state. There has also been excluded a number of large floods when there has been a record of a larger flood close by on the same river.

Floods from Table 1 having values of C in Eqs. 4 and 4a greater than 30 are plotted in Fig. 1. These data are in terms of the maximum momentary flow. In the original records there were a few floods which were published in terms of 24-hour average flow and these have been adjusted for momentary flow by Fuller's equation:[3]

$$Q = Q_1(1 + 2A^{-0.3}) \qquad [1]$$

where Q_1 = the recorded 24-hour average flood in cubic feet per second;
Q = the corresponding momentary peak flow;
A = the drainage area in square miles.

This equation is admittedly approximate, having been based on very few observations. However, it is the only one we have and it has been widely used. On pages 97 to 113 of Ref. 26 of Art. 65 is given a tabulation of the momentary peaks in terms of the reported maximum calendar-day average and the maximum 24-hour average for 690 floods; but unfortunately, the paper states that time was insufficient to draw any conclusions. It shows the ratio to be rather erratic, depending as it does upon the nature of the storm and, as between rivers, upon the storage on the watershed and other things.

The plotted points in Fig. 1 disregard certain published great floods whose records are unqualified guesses and are probably far from accurate. These are marked with a † in Table 1 and have been listed solely because they have been used by others in previous plottings.

When the magnitude of the peak of a flood from a given drainage area is known, it is sometimes desired to know what the peak probably would be from

[1] C. S. JARVIS, "Flood Flow Characteristics," *Trans. Am. Soc. Civil Engrs.*, 1926, p. 985, assisted by W. S. Eisenlohr and R. S. Goodridge.

[2] W. P. CREAGER and J. D. JUSTIN, Hydro-Electric Handbook, John Wiley & Sons, 1927.

[3] W. E. FULLER, "Flood Flows," *Trans. Am. Soc. Civil Engrs.*, 1914, p. 564.

TABLE 1

Unusual Flood Peaks, United States and Foreign Rivers

By C. S. Jarvis and William P. Creager

* Items with asterisk have been stepped up from 24-hr average flood to estimated momentary peak by equation 1.

† Items with dagger are unqualified (see text) and are not plotted on Fig. 1.

( ) Items in parentheses are known to be approximate or unofficial.

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
	Alabama					
1	Tennessee R., Florence	30,800	444,000	14.4	Mar. 1897	132
2	Tennessee R., Decatur	26,300	283,000	10.8		142
3	Alabama R., Selma	15,400	146,000	9.5	Jan. 1892	1
4	Coosa R., Childersburg	8,390	150,000	17.9	July 1916	99
5	Black Warrior, Tuscaloosa	4,830	215,000	44.6	Apr. 18, 1900	155
6	Tallapoosa, Milstead	3,840	70,000	18.2	Dec. 1901	72
7	Tallapoosa, Sturdevant	2,500	59,000	23.6	Mar. 1906	69
8	Elk, Rogersville	2,100	61,600	29.4		142
9	Black Warrior, Cordova	1,900	57,000	30.0	Mar. 1902	120
10	Elk R., Elkmont	1,700	51,800	30.4		142
11	Conecuh R., Brantley	504	15,600	31.0	Aug. 19, 1939	191
12	Choccolocco Cr., Jenifer	272	11,800	43		69
13	Camp Branch, Ensley	7.4	510	69	1909	11
14	Venison Branch, near Mulga	3.9	207	53		11
15						
16						
17						
18						
	Arizona					
19	Colorado R. below Gila Junction	225,000	236,000	1.05	Jan. 1916	119
20	Gila R., Yuma	56,000	220,000	3.93	Jan. 1916	75
21	Gila R., Florence	17,750	133,000	7.5	Feb. 1891	72
22	Salt R., below Phoenix	12,000	296,000	24.7	Feb. 1891	72
23	Salt R., McDowell	6,260	138,000	22.0	Mar. 1893	69
24	Verde R., McDowell	6,000	166,000*	27.6	1893	72
25	Salt R., Roosevelt	5,756	207,000	36.0	Mar. 1893	72
26	San Pedro R., near Mammoth	3,850	90,000	23.4	Sept. 28, 1926	167
27	San Pedro R., Charleston	1,480	98,000	66.3	Sept. 28, 1926	167
28	Canyon Diablo, Leupp	544	44,600	82	Sept. 1923	74
29	Troxton Canyon, E. of Kingman	450	49,500	110	1894	2
30	Canyon Diablo, Arch Bridge	340	35,400	104	Sept. 1923	74
31	Sonoita Cr., near Patagonia	210	20,000	95.4	Aug. 1934	167
32	Cave Cr., near Phoenix	200	25,000	125	Aug. 1921	91
33	Pinal Cr., Globe	30	13,200	440	Aug. 1904	91
34	Chase Cr., of Gila River	20	12,940	647	Dec. 1906	91
35						
36						
37						
38						
39						
	Arkansas					
40	Mississippi R., above Arkansas Junction	1,050,000	2,420,000	2.31	1912	89
41	Mississippi R., Helena	1,000,000	2,040,000	2.04	1912	119

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
42	Arkansas R., Van Buren	150,300	552,000 (600,000)	3.7 (4.0)	Apr. 16, 1927 Oct. 1941	132 195
43	Red R., Garland	51,500	327,000	6.35	Feb. 25, 1938	132
44	White R., Clarendon	19,000	320,000	16.8	1916	89
45	Ouachita R., Remmel Dam	1,540	140,000	91	May 16, 1923	132
46						
47						
48						
49						
50						
	California					
51	Sacramento R.	22,500	575,000	25.6	1904	2
52	Sacramento R., Red Bluff	9,300	296,000	31.8	Dec. 1937	161
53	Feather R., Oroville	3,627	187,000	51.6	Mar. 1907	70
54	Eel R., Scotia	3,070	290,000	95	Feb. 1915	132
55	Feather R., N. Fk., Big Bend	1,940	109,000	56.3	Mar. 1907	8
56	American R., Fair Oaks	1,921	182,900	95	Mar. 1928	132
57	Yuba R., Smartville	1,201	120,000	99.9	Mar. 26, 1928	171
58	Los Angeles R., Long Beach	1,060	80,000	75	Mar. 2, 1938	162
59	Santa Ana R., Mentone	845	100,000	118	Mar. 2, 1938	198
60	Putah Cr., Winters	655	60,000	91.6	Dec. 1913	70
61	American R., Middle Fk., near E. Auburn	619	100,000	162	Mar. 25, 1928	171
62	Smith R., near Crescent City	613	61,700	101	Mar. 18, 1932	171
63	McCloud R., near Gregory	608	53,700*	88.3	Mar. 1904	69
64	San Luis Rey R., Oceanside	565	95,500	169	Jan. 1916	66
65	Los Angeles R., Dayton Ave.	510	68,000	133	Mar. 2, 1938	180
66	San Luis Rey R., Bonsall	465	128,000	275	Feb. 23, 1891	169
67	Calaveras R., Jenny Lind	394	69,500	176	Jan. 1911	70
68	San Diego R., Santee	375	70,300	187	Jan. 1916	66
69	San Luis Rey R., near Pala	325	75,000	231	Jan. 1916	70
70	San Dieguito R., Bernardo	299	72,200	241	Jan. 1916	66
71	Bear R., Van Trent	262	87,800	336	Feb. 1907	70
72	Sespe Cr., near Fillmore	254	56,000	220	Mar. 2, 1938	198
73	Mattole R., New Petrolia	249	55,600	223		70
74	Smith R. at (Junction) Crescent City	227	42,500	187	Nov. 1915	70
75	San Gabriel R., Azusa	222	55,000	248	Mar. 2, 1938	162
76	Santa Ynez R., near Santa Barbara	219	38,174	174	Mar. 2, 1938	198
77	San Luis Rey R., Mesa Grande	209	58,500	280	Jan. 1916	66
78	San Gabriel R., Dam No. 1—Inflow	204	90,000	440	Mar. 1938	192
79	San Diego R., Lakeside	189	38,000	201	Jan. 1916	66
80	Sweetwater R., Jamacha	172	43,000	250	Jan. 1916	66
81	San Jacinto R., near San Jacinto	140	45,000	322	Feb. 16, 1927	171
82	Sweetwater R., near Dehesa	112	24,300	217	Jan. 1916	66
83	San Jacinto R., near San Jacinto	108	30,000	278	Jan. 1916	66
84	Otay R., Lower Otay Dam	98.6	37,400	379	Jan. 1916	70
85	Putah Cr., near Guenoc	91.0	24,600	270	Mar. 1904	91
86	Los Angeles R., Tujunga No. 1 Dam	81.4	34,000	418	Mar. 2, 1938	180
87	Smith R., N. Fk., Crescent	81.0	28,200	348	Nov. 1915	99

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
88	Santa Ysabel Cr., Mesa Grande	53.4	21,100	395	Jan. 1916	66
89	Lytle Cr., near Fontana	47.9	25,200	526	Mar. 2, 1938	198
90	San Gabriel R., San Gabriel Dam No. 2	40.4	23,800	588	Mar. 2, 1938	164
91	Santa Paula Cr., Ventura County	39.8	13,500	339	Mar. 2, 1938	198
92	Pine Tree Canyon, 12 mi N. of Mojave	35.0	59,500	1,700	Aug. 12, 1931	148
93	Little Tujunga Cr., Canyon Mouth, near Los Angeles	19.3	8,540	442	Mar. 2, 1938	180
94	Topanga Cr., near Topanga Beach	17.9	7,960	444	Mar. 2, 1938	198
95	Arroyo Seco, 5.5 miles N. W. of Pasadena	16.4	8,630	526	Mar. 2, 1938	198
96	San Gabriel R., Devil's Canyon above Dam No. 2	15.4	23,000	1,490	Mar. 2, 1938	200
97	Santa Anita Canyon, Santa Anita Dam	10.8	4,600	425	Mar. 2, 1938	180
98	Sawpit Canyon, Los Angeles	7.4	4,070	550	1889	107
99	Cameron Cr., near Tehachapi	3.59	13,500	3,760	Sept. 30, 1932	201
100	Fall Cr., near mouth, near Los Angeles	2.2	4,200	1,880	Mar. 2, 1938	180
101	Upper Willow Springs Canyon, near Mojave	0.81	4,900	6,050	Sept. 30, 1932	201
102						
103						
104						
105						
106						
	Colorado					
107	Colorado (Grand) R., Fruita	17,100	120,000	7	July 1884	99
108	Arkansas R., Pueblo	4,600	102,700	22.3	June 1921	68
109	Arkansas R., Pueblo	1,740	99,300	57.0	June 1921	91
110	Bijou Cr., at mouth	1,444	283,000	196	May 31, 1935	139
111	Republican R., Newton	1,270	103,000	81.1	May 1935	152
112	Arkansas R., Florence to Pueblo	940	75,200	80	June 1921	106
113	Republican R., S. Fk., Newton	669	83,000	124	May 30, 1935	139
114	Purgatoire Cr., Nine Mile Dam	635	64,200	101	Sept. 15, 1934	139
115	St. Charles R., Pueblo	482	71,800	149	June 1921	68
116	W. Bijou Cr., Byers	280	164,670	588	May 30, 1935	139
117	Kiowa Cr., Bennett	266	75,300	284	May 30, 1935	139
118	Middle Bijou Cr., Peoria	230	143,640	623	May 30, 1935	139
119	Kiowa Cr., N. of Kiowa	190	110,000	578	May 30, 1935	139
120	Middle Bijou Cr., below Wilson Cr.	151	71,270	473	May 1935	139
121	Cherry Cr., Castlewood, Dam	131	32,000	244	Aug. 2, 1933	139
122	Monument Cr., Colorado Springs	130	50,000	385	May 30, 1935	139
123	W. Bijou Cr., Johnson's Bridge	118	34,250	291	May 30, 1935	139
124	Horse Cr., near Holly	100	22,000	220	Aug. 28, 1935	139
125	Dry Cr., near Pueblo	86	24,300	283	June 1921	68
126	Kiowa Cr., Elbert	60	43,500	725	May 30, 1935	139
127	Rock Cr., near Pueblo	59.0	53,800	913	June 1921	68

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
128	Granada Cr., above Granada	40	31,000	775	July 11, 1935	139
129	Peck's Cr., near Pueblo	34.4	19,400	564	June 1921	68
130	Burro Canyon, Madrid	29.0	24,800	860	1925	170
131	Boggs Cr., near Pueblo	26.0	15,100	582	June 1921	68
132	N. Arroyo, near Pueblo	15.6	9,660	619	June 1921	68
133	Osteen Arroyo, near Pueblo	7.8	9,050	1,160	June 1921	68
134	Cameron Arroyo, near Pueblo	7.3	13,900	1,900	June 1921	68
135	Templeton Gap, Colorado Springs	7.1	6,120	862	May 1922	76
136	Blue Ribbon Cr., Pueblo	6.7	9,110	1,360	June 1921	68
137	Hogan's Gulch, Eden	6.1	9,640	1,580	Aug. 1904	112
138	Missouri Canyon, near mouth, Sec. 26, T.6N, R.70W.	2.4	4,350	1,810	June 15, 1923	135
139	S. Arroyo, near Pueblo	1.8	1,910	1,060	June 1921	68
140	Magpie Gulch, near Golden	1.5	1,910	1,270	July 1923	112
141	Skyrocket Cr., Ouray	1.0	2,000	2,000	July 1923	112
142						
143						
144						
145						
146						
	Connecticut					
147	Connecticut R., Thompsonville	9,637	282,000	29.3	Mar. 20, 1936	153
148	Hoosatonic, Gaylordsville	1,020	31,600	31.0		69
149	Farmington R.	584	24,400	41.7		72
150	Scantic R., N. Br.	118	6,140	52		72
151	Hockanum R.	79.0	6,160	78		72
152	Farmington R., E. Br., West Hartford	47.4	6,720	141	Nov. 1927	158
153	Pequonnock, Bridgeport	25.0	3,920	157	July 1905	1
154						
155						
156						
157						
158						
	District of Columbia					
159	Potomac R., near Washington	11,560	484,000	42.0	Mar. 1936	135
160	Rock Cr., Q St., N. W., Washington	77.5	9,765	126		140
161	Rock Cr., at Sherill Drive, Washington	62.2	4,460	71.7	Aug. 1933	172
162						
163						
164						
165						
166						
	Florida					
167	Yellow R., near Holt	1,220	34,400* (90,000)	28.2 (73.8)	Aug. 19, 1939 1929	191 191

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
168	Alafia R.	336	30,000	89.3		182
169						
170						
171						
172						
173						
	Georgia					
174	Appalachicola R., Junction	17,300	381,000	22		74
175	Savannah R., Augusta	7,304	350,000	48	Oct. 3, 1929	132
176	Ocmulgee R., Lumber City	5,180	43,800	8.5	Mar. 9, 1939	191
177			(150,000)	(29.0)	Jan. 21, 1925	191
178	Chattahoochee, West Point	3,300	139,000	42.1	Dec. 1919	99
179	Oconee Milledgeville	2,840	115,000	40.5	Jan. 1925	119
180	Rhine, Macon	2,574	96,300	37.4		69
181	Ocmulgee R., Macon	2,425	90,000	37.1	Jan. 1925	119
182	Flint R., Culloden	2,000	85,000	42.5	July 1916	99
183	Etowah R., Rome	1,800	59,400	33.0	Mar. 1906	69
184	Oconee R., Greensboro	1,100	68,200	62	Aug. 1908	119
185	Broad R., near Carlton	762	47,200	62	Aug. 1908	90
186	Tocca R., near Blueridge	231	12,200	53	1901	16
187	Soquee R., Demorest	158	8,850	56		69
188						
189						
190						
191						
192						
	Idaho					
193	Snake R., near Murphy	41,900	47,400	1.13	June 1918	103
194	Snake R., S. Fk., Minidoka	22,600	54,200	2.40	June 1896	69
195	Salmon R., Whitebird	13,400	120,000	9.0	June 1894	173
196	Snake R., S. Fk., Lyon	5,480	51,500	9.4	May 1904	69
197	Clearwater R., Kamiah	4,850	76,600	15.8	May 1913	72
198	Payette R., Horshoe Bend	2,230	22,100	9.9	June 1921	173
199	Weiser R., Weiser	1,670	17,900	10.7	May 1896	99
200	Coeur d'Alene R., near Cataldo	1,220	22,000	18.0	Mar. 1921	103
201	St. Joe R., Calder	1,080	17,300	16.0	May 1922	103
202	Teton R., near St. Anthony	960	7,590	7.9	June 1909	69
203	Clearwater R., S. Fk., Grangeville	940	9,870	10.5	May 1912	103
204	Mooyie R., Snyder	717	10,800	15.1	June 1916	103
205	Payette R., N. Fk., Van Wyck	586	8,800	15	May 1921	103
206	St. Maries R., Lotus	420	8,830	21	Mar. 1921	103
207	Payette R., N. Fk., Lardo	131	4,190	32	June 1909	103
208	Hull's Gulch, Boise	5.0	5,000	1,000	July 1913	91
209						
210						
211						
212						
213						
	Illinois					
214	Mississippi R., Cairo	902,900	2,010,000	2.23	1912	89
215	Ohio R., Cairo	203,900	1,950,000	9.56	Feb. 3–4, 1937	149

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
216	Wabash R., Mt. Carmel	28,600	428,000	15.0	Mar. 30, 1913	132
217	Illinois R., at mouth	27,914	125,000	4.48	Apr. 1904	90
218	Illinois R., Peoria	13,480	80,100	5.94	Mar. 1904	72
219	Kankakee R., Custer Park	4,870	33,700	6.9	May 14, 1933	157
220	Iroquois R., Chebanse	2,120	27,000	12.7	May 13, 1933	157
221	Spoon R., Seville	1,600	35,300	22.0	Aug. 22, 1924	157
222	Pecatonica R., Freeport	1,330	18,400	13.8	Mar. 16, 1929	157
223	Mackinaw R., Green Valley	1,100	21,800	19.8	May 19, 1927	157
224	Vermilion R., Streator	1,080	16,000	14.8	Jan. 21, 1916	157
225	Big Muddy R., Plumfield	753	16,300	21.6	Feb. 1, 1916	157
226	Des Plaines R., Riverside	630	13,100	20.8	1889	72
227	Sangamon R., S. Fk., Kincaid	510	11,800	23.2	Mar. 16, 1922	157
228	Spring Cr., Joliet	19.7	1,070	54.3	June 11, 1926	157
229						
230						
231						
232						
233						
	Indiana					
234	White R., Hazleton	11,300	235,000	20.8	Mar. 29, 1913	132
235	White R., E. Fk., Shoals	4,940	136,000	27.5	Mar. 28, 1913	132
236	Wabash R., Logansport	3,760	116,000	30.9	Mar. 26, 1913	132
237	Antietam Cr., Sharpsburg	295	6,790	23	1902	17
238	Gunpowder Falls, Glencoe	160	5,600	35		69
239						
240						
241						
242						
243						
	Iowa					
244	Missouri R., Sioux City	323,462	531,000	1.64	1881	30
245	Mississippi R., Keokuk	119,000	360,000	3.02	1851	150
246	Mississippi R., Clayton	79,040	210,000	2.66	1880	30
247	Des Moines R., Keosauqua	13,900	97,000	7.0	1903	174
248	Cedar R., Cedar Rapids	6,320	56,900	9.0	Apr. 1917	90
249	Iowa R., Iowa City	3,230	36,200	11.2	June 1918	174
250	Devil's Cr., near Viele	143	85,800	600	June 1905	9
251	Dry Run, Decorah	22.3	16,100	720	Mar. 1915	91
252	Little Devil's Cr.	19.0	10,600	560	June 1905	91
253	Panther Cr.	14.0	7,280	520	June 1905	91
254						
255						
256						
257						
258						
	Kansas					
259	Kansas R., Lawrence	59,841	228,000	3.80	1903	1
260	Kansas R., Junction City	44,910	179,000	4.0	May–June 1935	152

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
261	Republican R., Junction City	24,960	168,000	6.7	May–June 1935	152
262	Blue R., near Manhattan	9,490	86,600	9.13	May 1903	72
263	Neosho R., Iola	3,670	74,500	20.3	July 1904	2
264	Verdigris R., Liberty	3,067	50,300	16.4	July 1904	2
265	Cherryvale Cr., Cherryvale	2.0	1,860	930		31
266						
267						
268						
269						
270						
	Kentucky					
271	Mississippi R., Columbus	921,900	(2,500,000)	(2.71)	Feb. 27, 1937	135
272	Ohio R., Paducah	202,700	1,850,000	9.1	Feb. 1913	135
273	Ohio R., Louisville	90,600	1,100,000	12.1	Jan. 27, 1937	135
274	Ohio R., Ashland	60,600	740,000	12.2		165
275	Green R., Livermore	7,500	208,000	27.7	Jan. 27, 1937	132
276	Kentucky R., Lockport	6,170	99,000	16.0	Jan. 24, 1937	132
277	Cumberland R., Burnside	4,890	164,000	33.6		142
278	Licking R., Catawba	3,320	86,200	26.0	Jan. 23, 1937	132
279	Big Sandy R., Levisa Fk., Paintsville	2,150	69,000	32.1	Jan. 29, 1918	132
280	Cumberland R., Cumberland Falls	2,010	59,600	29.6		142
281	Cumberland R., S. Fk., Nevelsville	1,260	160,000	127	Mar. 23, 1929	132
282	Cumberland R., Barbourville	982	40,100	40.8		142
283	Rock Castle R., Rock Castle Springs	746	36,400	48.8		142
284						
285						
286						
287						
288						
	Louisiana					
289	Mississippi R., Carrollton	1,400,000	1,500,000	1.07	May 1922	89
290	Mississippi R., Red River Landing	1,242,700	(2,000,000)	(1.61)	Feb. 18, 1937	135, 132
291	Atchafalaya R., Krotz Springs	150,000	443,000	2.95	Feb. 28, 1937	132
292						
293						
294						
295						
296						
	Maine					
297	St. John R., Van Buren	8,270	134,000	16.2	May 2, 1923	175
298	Penobscot R., Bangor	7,700	115,000	14.9		10
299	St. John R., below Fish River at Fort Kent	5,690	121,000	21.3	May 5, 1933	154
300	Penobscot R., West Enfield (Montague)	4,690	153,000	32.6	May 1, 1923	99
301	Kennebec R., Waterville	3,030	157,000	51.1	Dec. 16, 1901	99

TABLE 1—*Continued*

Unusual Flood Peaks, United States and Foreign Rivers

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
302	Androscoggin R., Gulf Island	2,260	149,000	66	Mar. 19, 1936	146
303	Saco R., W. Buxton	1,572	80,000	51	Mar. 22, 1936	146
304	Mattawaumkeag R., Mattawaumkeag	1,500	43,900	29.2	May 1, 1923	175
305	Androscoggin R., Rumford	1,248	68,300*	54.7	Apr. 15, 1895	99
306	Piscataquis R., near Foxcroft	286	21,700	75.8	Sept. 29, 1909	154
307						
308						
309						
310						
311						
	Maryland					
312	Potomac R., Point of Rocks	9,654	480,000	50	Mar. 1936	135
313	Potomac R., Cumberland	875	85,000	97	Mar. 1936	146
314	Monocacy R., Jag Bridge, near Frederick	817	64,700*	79.2	Aug. 24, 1933	99
315	Gunpowder R.	302	25,100	83	1889	69
316	Potomac R., Bloomington	287	74,900	261	Mar. 23, 1924	146
317	Wills Cr., Cumberland	247	43,700	177	Mar. 1936	172
318	Octoraro Cr., near Rising Sun	191	25,400	133	Aug. 24, 1933	166
319	Patapsco R., N. Br., near Marriottsville	165	19,500	118	Aug. 24, 1933	99
320	Town Cr., near Oldtown	148	27,000	182	Mar. 1936	187
321	Deer Cr., Rocks	94.4	22,600	239	Aug. 23, 1933	166
322	Lake Roland	39.0	8,970	230	1868	91
323	Little Gunpowder Falls, Laurel Branch	36.1	9,200	255	Aug. 23, 1933	99
324	Anacostia R., N. W. Br., near Colesville	21.3	4,500	211	Aug. 23, 1933	99
325	Owens Cr., Lantz	5.7	4,500	790	Dec. 1934	187
326						
327						
328						
329						
330						
	Massachusetts					
331	Connecticut R., Montague City	7,840	236,000	30.1	Mar. 19, 1936	153
332	Merrimack R., Lowell	4,424	173,000	39.1	Mar. 20, 1936	153
333	Westfield R., near Westfield	497	55,500	112	Sept. 21, 1938	183
334	Deerfield R., Charlemont	362	56,000	155	Sept. 21, 1938	183
335	Great R., Westfield	350	52,800	151	1878	72
336	Westfield R., Knightville	162	33,700	208	Sept. 21, 1938	183
337	Fomer R., above reservoir, Holyoke	13.0	2,840	218		69
338	Manhan R., Holyoke	13.0	2,370	182	Feb. 1900	91
339						
340						
341						
342						
343						

TABLE 1—*Continued*

Unusual Flood Peaks, United States and Foreign Rivers

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
	Michigan					
344	Grand R., Grand Rapids	4,900	49,500	10.1	June 1905	72
345	Tittabawassee R., Freeland	2,461	49,500	20.1	Mar. 1919	102
346	Escanaba R., Escanaba	800	10,700	13.4		69
347	Dead R., Forestville	142	2,420	17		69
348						
349						
350						
351						
352						
	Minnesota					
353	Mississippi R., St. Paul	36,800	107,000	2.91	Apr. 29, 1881	132
354	Mississippi R., Anoka	17,100	49,100	2.87		25
355	Minnesota R., Mankato	14,600	43,800	3.0		25
356	Mississippi R., Sauk Rapids	12,400	50,900	4.1		69
357	St. Croix R., St. Croix Falls	5,950	35,700	6.0		72
358	Pine R., below Pine Reservoir	452	18,100	40	June 1908	69
359						
360						
361						
362						
363						
	Mississippi					
364	Mississippi R., Vicksburg	1,144,500	2,495,000	2.18	May 4, 1927	132
365	Yazoo R., Yazoo Mouth	13,850	139,000	10.0	Apr. 1874	72
366	Coldwater R., Coldwater	1,400	60,000	42.8	Jan. 21, 1935	160
367	Tombigbee R., E. Fk., near Fulton	650	24,200 (80,000)	37.2 (123)	Feb. 15, 1939 March 1927	191 191
368	Rocky Cr., near Ellisville	15.0	16,600	1,110	May 1882	91
369						
370						
371						
372						
373						
	Missouri					
374	Mississippi R., St. Louis	701,000	1,300,000	1.85	June 28, 1844	132
375	Missouri R., St. Charles	530,810	600,000	1.13	June 19, 1844	30
376	Osage, Bagnell	14,000	150,000	10.7	June 1844	196
377	Meramec R., Eureka	3,800	175,000	46.1	Aug. 22, 1915	132
378	Big R., Byrnesville	892	80,000	89.7	Aug. 1915	132
379	Castor R., Zalma	395	40,000	102	Jan. 14, 1937	132
380	Rio des Perca. St. Louis	23.8	6,090	256	Aug. 1915	81
381	Rio des Perca, near St. Louis	15.6	6,400	410	Aug. 1915	81
382						
383						
384						
385						
386						

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
	Montana					
387	Yellowstone R., Intake	66,800	159,000	2.38	June 1921	98
388	Clark Fk., near Plains	19,900	115,000	5.78	June 1913	98
389	Kootenai R., Libby	11,000	130,000	11.8	June 1916	98
390	Flathead R., near Polson	7,010	75,000	10.7	June 1913	98
391	Flathead R., Columbia Falls	4,560	88,000	19.3	June 1922	98
392	Flathead R., N. Fk., Belton	900	48,600	54	June 1916	98
393	Sun R., N. Fk., Augusta	600	32,400	54	June 1916	68
394	Beaver Cr., Wibeaux	311	33,000	106	June 7, 1929	197
395	Custer Cr., N. E. of Miles City	155	21,000	135	June 19, 1938	135
396	Le Noir Coulee, Malta	16	8,610	538	June 1906	120
397						
398						
399						
400						
401						
	Nebraska					
402	Republican R., Cambridge	12,300	280,000	22.8	May 1935	152
403	Republican R., Max	5,840	190,000	32.5	May–June 1935	152
404	Republican R., below Benkelman	5,134	190,000	37	May–June 1935	169
405	North Loup, St. Paul	4,020	76,400	19	June 1899	120
406	Republicau R., Kansas state line	2,550	150,000	58.8	May 1935	152
407						
408						
409						
410						
411						
	Nevada					
412	Humboldt R., Oreana	13,800	3,040	0.22	May 1897	72
413	Meadow Valley Wash, near Moapa	2,150	8,170	3.8	Jan. 1910	74
414	Truckee R., Reno	1,070	7,490	7.0	1913	72
415	Carson R., E. Fk., Rodenbohs	414	5,380	13		69
416	Carson R., E. Fk., State line	298	4,880*	16.4	June 1911	70
417	Baker Cr., Baker	10.0	170	17	1914	72
418						
419						
420						
421						
422						
	New Hampshire					
423	Connecticut R., Orford	3,100	57,400	18.5	Mar. 1913	99
424	Merrimac R., Franklin Junction	1,507	83,000	55.1	Mar. 19, 1936	136
425	Merrimac R., Franklin Junction	982	55,600	56.6	Nov. 1927	99
426	Pemigewassett R., Plymouth	622	65,800	106	Mar. 19, 1936	134
427	Saco R., near Conway	386	40,600	105	Mar. 19, 1936	153
428	Bakers R., near Wentworth	52	15,000	288	Nov. 1927	99
429	Peabody R., near Gorham	40	9,920	248	Nov. 1927	99
430	Ellis R., above Wildcat Brook, Jackson	28	14,800	529	Nov. 1927	99

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
431	Peabody R., near Glen House	17.4	7,330	421	Nov. 1927	99
432						
433						
434						
435						
436						
	New Jersey					
437	Delaware R., Trenton	6,796	295,000	43	Oct. 1903	187
438	Delaware R., Belvidere	4,542	220,000	48.5	Oct. 10–11, 1903	166
439	Raritan R., Bound Brook	806	66,000*	82	Sept. 1882	1
440	Passaic R., Paterson	785	35,600*	45.5	Oct. 10, 1903	166
441	Pompton R., Two Bridges	380	23,600	62	1903	1
442	Raritan R., N. Br., Milltown	190	15,600	82.1	Sept. 15, 1933	99
443	Ramapo R., Mahwah	118	12,500	106	Oct. 1903	120
444	Pequannock R., Macopin Dam	63.7	8,450*	133	Oct. 9, 1903	99
445	Raritan R., N. Br., near Far Hills	26	7,000	269	July 23, 1919	166
446						
447						
448						
449						
450						
	New Mexico					
451	San Juan R., at Ship Rock	12,800	150,000	11.7	Oct. 6, 1911	186
452	Canadian R., at Logan	11,200	278,000	24.8	Sept. 30, 1904	186
453	S. Canadian R., near Tucumcari	7,250	280,000	38.6	1904	184
454	Canadian R., at Taylor Springs	2,830	91,100	32.2	Sept. 1904	186
455	Ute Cr., near Logan	2,010	100,000	49.8	May 1, 1914	186
456	Canadian R., at French	1,480	156,000	105	Sept. 1904	186
457	Pecos R., near Anton Chico	1,080	40,300	37.3	June 1, 1937	186
458	Conchas R., at Variadero	690	51,800	75	June 3, 1937	186
459	Mora R., at Loma Parda	585	34,500	59	June 11, 1913	186
460	Mora R., Weber	294	27,600	94	Sept. 1904	68
461	Sapello Cr., at mouth, near Watrous	284	62,900	222	Sept. 29, 1904	186
462	Turquillo R., Mora Valley	160	16,000	100	1893	97
463	Mora R., below Mora	159	22,300	140	Sept. 1904	2
464	Gallinas R., at Montezuma	89	11,600	130	Sept. 30, 1904	186
465	Palomas R., near Hermosa	52	8,680	167	July 1925	74
466	Tanner Draw, near Clapham	20.3	11,200	552	May–June 1937	186
467	Draw, near Clayton	2.66	2,550	958	May–June 1937	186
468						
469						
470						
471						
472						
	New York					
473	St. Lawrence R., near Ogdensburg	298,080	319,000	1.07		105
474	Niagara R., Niagara	263,440	298,000	1.13		114
475	Niagara R. (Land Area Only)	175,000	299,000	1.71		114
476	Hudson R., Albany	8,100	220,000	27.2	Mar. 28, 1913	141
477	Hudson R., Mechanicsville	4,500	120,000	26.7	Mar. 28, 1913	166

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
478	Mohawk R., Cohoes	3,456	140,000	40.6	Mar. 1914	146
479	Delaware R., Port Jervis	3,076	155,000	50.4	Oct. 10, 1903	166
480	Chemung R., Chemung	2,530	92,300	36.5		166
481	Susquehanna R., Conklin	2,240	62,100	28	Mar. 18, 1936	146
482	Chemung R., below Big Flats	2,150	87,200	41	Mar. 1936	187
483	Chemung R., Elmira	2,055	138,000	67.2	June 1, 1889	9
484	Chenango R., Chenango Forks	1,492	82,800	55.5	July 1935	154
485	Tioga R., near Erwins	1,370	59,800	44	Mar. 12, 1936	146
486	Genesee R., St. Helena	992	43,600	44.0	May 1916	113
487	Schoharie Cr., Fort Hunter	900	49,600	55.1	Mar. 1901	37
488	Black R., Lyons Falls	897	41,300	46.0	Apr. 1869	72
489	Delaware R., E. Br., Hancock	838	91,800	110	Mar. 26, 1904	99
490	Delaware R., E. Br., Fishs Eddy	783	53,300	68.1	Aug. 24, 1933	166
491	Tioga R., Lindley	770	41,200	54	Mar. 1936	187
492	Tioughnioga R., Itaska	735	44,700	60.8	July 8, 1935	166
493	Delaware R., W. Br., Hales Eddy	593	46,000	77.6	Oct. 10, 1903	166
494	Cohocton R., near Campbell	472	45,400	96.2	July 8, 1935	166
495	Cattaraugus Cr., Versailles	467	29,900	64	Mar. 1918	104
496	Ausable R., Ausable Forks	444	24,900	56	Mar. 1913	113
497	Esopus Cr., Saugerties	417	55,100	132	Dec. 1878	69
498	West Canada Cr., Hinckley	372	39,100	105	Apr. 1869	37
499	Canisteo R., West Cameron	344	35,000	102	July 1935	99
500	Croton R., Croton Dam	339	25,400	75	1867	72
501	East Canada Cr., Dolgeville	264	20,000*	75.8	Mar. 26, 1913	99
502	Beaver Kill, Cook's Falls	241	19,000	79	Aug. 1933	187
503	Schoharie Cr., Prattsville	236	29,000	123	Sept. 1924	101
504	Neversink R., Oakland Valley	222	20,000	90	Aug. 24, 1933	133
505	Catskill Cr., South Cairo	210	21,000	100	Spring 1901	27
506	Esopus Cr., Coldbrook	192	55,000	286	Aug. 24, 1933	166
507	Owego Cr., near Owego	186	23,500	126	July 8, 1935	166
508	Canisteo R., Canisteo	185	25,000	135	July 1935	187
509	Fall Cr., Ithaca	124	25,800	208	July 1935	122
510	Ouleout Cr., East Sidney	101	16,700	165	July 1935	187
511	Rondout Cr., near Lackawack	100	26,715	267	Aug. 26, 1928	166
512	Salmon Cr., Myers	89.2	18,500	207	July 1935	133
513	Bennett Cr., near Canisteo	71.5	12,400	173	July 1935	99
514	Canacadea Cr., Hornell	59.4	26,600	448	July 1935	99
515	Taughannock Cr., N. Halseyville	56.7	42,100	742	July 1935	133
516	Canacadea Cr., Almond	49.8	22,000	442	July 1935	187
517	Meads Cr., E. Campbell	46.1	30,300	657	July 1935	133
518	Campbell Cr., near Kanona	35.8	14,000	391	July 1935	99
519	Dudley Cr., near Lisle	29.6	16,200	547	July 1935	133
520	Glen Cr., Watkins Glen	21.3	27,900	1,310	July 1935	133
521	Purdy Cr., near Canisteo	21.2	8,990	424	July 1935	99
522	Merrill Cr., near Upper Lisle	20.8	15,100	726	July 1935	99
523	Stony Brook, Stony Brook Glen	18.1	5,800	320	July 1935	99
524	Fivemile Cr., Enfield	18.0	8,380	466	1935	133
525	Big Cr., near North Hornell	16.5	11,900	721	July 1935	99
526	Sawkill, near Bearsville	12.1	9,980	825	July 1935	99
527	Trumansburg Cr., Trumansburg	11.5	17,800	1,550	July 1935	133
528	Willet Cr., Marathon	11.0	6,430	585	July 1935	99

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
529	Sawkill, near Shady	9.5	9,180	966	July 1935	133
530	Stephens Cr., near Carson	7.04	6,700	952	July 1935	99
531	Strongs Br., near Smithville Flats	6.41	6,650	1,040	July 1935	99
532	Pine Cr., near Monterey	5.0	3,270	654	July 1935	187
533	Glen Cr., near Townsend	2.91	7,330	2,520	July 1935	133
534	Harrisburg Hollow, near Hickory Hill	2.49	2,810	1,130	July 1935	99
535	Brook, Bradford	1.68	1,940	1,150	July 1935	99
536	Mad Cr., Leroy	1.5	3,450	2,300	May 1916	71
537	Gilmore Br., near Preston	0.62	518	835	July 1935	99
538	Beacon Cr., near Fishkill	0.25	800	3,200	July 1897	27
539						
540						
541						
542						
543						
	North Carolina					
544	Roanoke R., Old Gaston	8,350	275,000	32.9	Nov. 26, 1877	99
545	Pee Dee R., near Rockingham	6,910	212,000	30.7	Sept. 19, 1928	155
546	Cape Fear R., Fayetteville	4,290	133,000	31.0	Aug. 29, 1908	156
547	Yadkin R., High Rock	3,930	184,000	46.8	July 1916	99
548	Yadkin R., Donaha	1,600	80,000	50	July 1916	99
549	Haw R., near Pittsboro	1,340	98,000	73.7	Aug. 1908	156
550	French Broad R., Asheville	949	90,000	94.8	July 1916	99
551	Little Tennessee R., Jackson	675	57,500	85.3	Dec. 1901	21
552	Tuckasegee R., Bryson	662	38,600	58.2	Mar. 1889	72
553	Flat R., Bahama	150	13,600	90.7	Sept. 8, 1934	176
554	Broad R., near Chimney Rock	97	20,500	211	Aug. 15, 1928	176
555	Little Sugar Cr., near Charlotte	41.4	7,030	170	Aug. 16, 1928	176
556	Morgan Cr., near Chapel Hill	27	30,000	1,110	Aug. 4, 1924	196
557	Cane Cr., Bakersville	22.0	29,500	1,340	May 1901	34
558	Pigeon R., W. Fk., Spruce	12.2	16,500	1,350	Aug. 1940	194
558a	Pigeon R., Middle Prong, Spruce	8.4	16,400	1,950	Aug. 1940	194
559	Big Cr., near Sunburst	1.69	12,400	7,340	Aug. 30, 1940	189
560	Big Cr., near Sunburst	1.32	12,900	9,800	Aug. 1940	194
561						
562						
563						
564						
565						
	North Dakota					
566	Red R., Grand Forks	25,000	42,500	1.70	1897	25
567	Little Missouri, Medora	5,780	19,100	3.3		69
568	Heart R., Richardton	1,250	8,000	6.4		72
569	Grande, N. Br., Haley	500	5,800	11.6	1913	72
570						
571						
572						
573						
574						

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
	Ohio					
575	Ohio R., Cincinnati	75,800	950,000	12.5	Jan. 26, 1937	135
576	Muskingum R., McConnellsville	7,410	270,000	37		135
577	Miami R., Miami	3,937	386,000	98	Mar. 1913	107
578	Scioto R., Chillicothe	3,850	250,000	65	Mar. 1913	132
579	Miami R., Dayton	2,510	250,000	100	Mar. 1913	132
580	Scioto R., Columbus	1,624	138,000	84.9	Mar. 25, 1913	147
581	Lower Scioto R., Columbus	1,570	111,000	70.5	Mar. 1913	71
582	Little Miami R., Milford	1,195	82,900	69.4	Mar. 19, 1933	132
583	Miami R., Tadmor	1,130	127,000	113	Mar. 1913	107
584	Scioto R., Columbus	1,047	84,800	80.8	Mar. 1913	39
585	Mad R., Osborn	649	76,000	117	Mar. 1913	107
586	Stillwater R., Englewood	646	85,400	132	Mar. 1913	147
587	Olentangy R., Columbus	514	50,400	98	Mar. 1913	71
588	Stillwater R., Sugar Grove	448	51,500	115	Mar. 1913	107
589	Twin Cr., Germantown	270	65,900	244	Mar. 1913	107
590	Ludlow Cr., above Dayton	65	17,300	266	Mar. 1913	91
591	Lost Cr., above Dayton	52	29,700	571	Mar. 1913	91
592	Honey Cr., E. Fk., New Carlisle	11.8	15,100	1,280	July 1918	71
593	Honey Cr., E. Fk., New Carlisle	6.7	14,800	2,210	July 1918	71
594	Honey Cr., W. Fk., New Carlisle	3.5	3,500	1,000	July 1918	109
595						
596						
597						
598						
599						
	Oklahoma					
600	Arkansas R., Muskogee	96,800	243,000 (270,000) (300,000)	2.5 (2.8) (3.1)	June 9, 1935 June 1923 Oct. 1941	132 132 195
601	W. Quartermaster Cr.	108	69,000	640	Apr. 1934	169
602	W. Quartermaster Cr.	61	34,200	560	Apr. 1934	169
603	Ninemile Cr.	42	36,100	860	Apr. 1934	169
604	E. Quartermaster Cr.	41.5	54,800	1,320	Apr. 1934	169
605	Sergeant Major Cr.	37	53,650	1,450	Apr. 1934	169
606	East Hay Cr. (Washita Basin)	4	6,400	1,590	Apr. 1934	169
607						
608						
609						
610						
611						
	Oregon					
612	Columbia R., Dalles	237,000	1,390,000	5.87	June 1894	72
613	Willamette R., Albany	4,860	303,000	62.2	1861	72
614	Willamette R., Middle Fk., Jasper	1,450	93,000	64.2		69
615	Siletz R., Siletz	204	40,800	200	Nov. 20, 1921	177
616	Willow Cr., near Heppner	125	36,000	288	1903	19
617	Willow Cr., near Heppner	20	36,000	1,800	June 4, 1903	19
618						
619						
620						
621						
622						

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
	Pennsylvania					
623	Susquehanna R., McCall's Ferry	26,800	870,000	32.5	Mar. 19, 1936	135
624	Ohio R., Pittsburgh	19,106	640,000	34	Mar. 1936	146
625	Susquehanna R., Sunbury	18,300	530,000	29	Mar. 1936	187
626	Susquehanna R., Danville	11,220	258,000	23	Mar. 18, 1865	154
627	Allegheny R., Kittaning	9,010	305,000*	33.8	Mar. 1913	90
628	Susquehanna R., Towanda	7,797	188,000	24.1	Mar. 1936 Mar. 17, 1865	166
629	Allegheny R., Parkers Landing	7,671	250,000	33	Mar. 1865	187
630	Susquehanna R., W. Br., Watsonton	6,596	284,000	43	Mar. 18, 1936	134
631	Allegheny R., Franklin	5,982	196,000	33	Mar. 1865	187
632	Susquehanna R., W. Br., Williamsport	5,682	313,000	55	Mar. 1936	146
633	Monongahela R., Lock No. 4	5,430	207,000	38.1	July 11, 1888	1
634	Juniata R., Newport	3,480	292,000	84	Mar. 1902	90
635	Susquehanna R., W. Br., Renovo	2,975	236,000	79.3	Mar. 18, 1936	135
636	Beaver R., Wampum	2,235	87,000	39	Mar. 1913	187
637	Schuylkill R., Philadelphia	1,893	127,000	67	Oct. 1869	187
638	Kiskiminetas R., Avonmore	1,723	200,000	116	Mar. 1936	134
639	Youghiogheny R., Sutersville	1,715	100,000	58	Mar. 1936	187
640	Youghiogheny R., Connellsville	1,326	92,500	70	Mar. 1936	187
641	Lehigh R., Bethlehem	1,280	94,000	73	Feb. 1902	187
642	Youghiogheny R., Ohiopyle	1,065	85,000	80	Mar. 1936	187
643	Juniata R., Raystown Br., Hawn's Bridge	948	86,500	91	Mar. 1936	187
644	Schuylkill R., Reading	900	80,100	89	1850	107
645	Juniata R., Frankstown Br., Petersburg	806	80,000	99	Mar. 1936	187
646	Juniata R., Raystown Br., Saxton	756	80,500	106	Mar. 1936	187
647	Conemaugh R., New Florence	748	91,200	122		134
648	Juniata R., Raystown Br., Juniata Crossing	549	67,000	122	Mar. 1936	187
649	W. Conewago Cr., near Manchester	510	47,600	93.3	Aug. 24, 1933	166
650	Stony Cr., Ferndale	451	58,600	130	Mar. 1936	187
651	Blacklick Cr., Blacklick	390	51,700	133	Mar. 1936	187
652	Clearfield Cr., Dimeling	371	37,600	211	Mar. 1936	187
653	Swatara Cr., Harper Tavern	333	53,000	159	June 1889	187
654	Juniata R., Frankstown Br., Williamsburg	291	47,600	164	Mar. 1936	187
655	Perkiomen Cr., Grater's Ford	279	41,200	148	July 1935	187
656	Loyalhanna Cr., New Alexandria	265	31,000	117	Mar. 1936	187
657	Codorus Cr., York	221	34,000	154	Aug. 23–24, 1933	135
658	Neshaminy Cr., near Langhorne	210	30,000	143	Aug. 1933	187
659	Sherman Cr., Shermandale	200	37,000	185	July 1927	187
660	Little Conemaugh R., Conemaugh	187	28,800	154	Mar. 1936	187
661	Pequea Cr., near Pequea	153	28,000	183	June 1938	187
662	Chester Cr., near Philadelphia	62	62,000	1,000	Aug. 1843	77
663	Darby Cr., near Philadelphia	48.0	27,800	580	Aug. 1843	77
664	Crum Cr., near Philadelphia	22	9,020	410	Aug. 1843	77
665	Ridley Cr., near Philadelphia	20	15,000	750	Aug. 1843	77

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
666	Mill Cr., Erie	12.9	12,900	1,000	Aug. 1915	91
667	Gist Run, near Dunbar	7.0	3,850	550	July 1912	187
668	Canodochly Branch, East Prospect	2.2	3,590	1,630	July 1914	71
669	Canodochly Cr., near Long Level	2.2	2,460	1,120	July 1914	187
670	Indian Run, Letort	2.1	4,060	1,930		71
671	Green Branch, Bridgeville	1.7	2,710	1,590	July 1914	83
672	Mann's Run, Creswell Station	0.67	1,700	2,540	July 1914	71
673	Docker's Hollow, North Braddock	0.60	2,400	4,000	June 1917	71
674	Whistler's Run, near Long Level	0.6	456	760	July 1914	187
675	Shingle Run, Johnstown	0.6	296	493	Aug. 1931	187
676	Bull's Run, Long Level	0.58	2,420	4,170	July 1914	71
677						
678						
679						
680						
681						
	Rhode Island					
682	Seekonk, Providence	190	10,800	57	1867	72
683	Flat River	61	7,320	120	Mar. 1843	72
684						
685						
686						
687						
688						
	South Carolina					
689	Santee R., Ferguson	14,800	368,000	24.8	July 22, 1916	176
690	Peedee R., Cheraw	9,100	273,000	30.0	Sept. 1908	74
691	Savannah R., Woodlawn	6,600	200,000	30.3	Aug. 26, 1908	99
692	Broad R. of Carolinas, Richtex	4,800	239,000	49.8	Oct. 3, 1929	156
693	Broad R., Alston	4,609	131,000	28.4	May 1901	72
694	Catawba R., near Rock Hill	3,050	151,000	50.0	May 23, 1901	99
695	Saluda R., near Silverstreet	1,570	83,800	53.4	Oct. 3, 1929	176
696	Catawba R., Catawba	1,535	110,000	71.7	July 1916	118
697	Pacolet R., Spartansburg	400	35,600	89	June 1903	19
698	Enoreé R., near Enoree	307	35,800	117	Oct. 2, 1929	176
699	Reedy R., near Princeton	215	28,000	130	Aug. 1908	99
700						
701						
702						
703						
704						
	South Dakota					
705	Cheyenne R., Hot Springs	8,720	150,000	17.2	May 1920	112
706	White R., near Interior	4,090	16,400	4.0	1905	72
707	Red Water R., Belle Fourche	1,006	8,050	8.0	1904	72
708						
709						
710						
711						
712						

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Author-ity
	Tennessee					
713	Mississippi R., Memphis	932,800	1,800,000	1.9	Jan. 29, 1937	135
714	Tennessee R., Johnsonville	38,500	460,000	11.9	Mar. 24, 1897	132
715	Tennessee R., Chattanooga	21,382	459,000	21.4	Mar. 11, 1867	135
716	Tennessee R., Breedenton	17,460	(400,000)	(22.9)	Mar. 11, 1867	135, 132
717	Cumberland R., Clarksville	16,000	290,000	18.1	Jan. 24, 1937	132
718	Cumberland R., Nashville	12,860	203,000	15.8	Jan. 1, 1927	132
719	Tennessee R., London	12,300	365,000	29.7	Mar. 11, 1867	135, 132
720	Cumberland R., Carthage	10,740	183,000	17.1	Dec. 30, 1926	132
721	Tennessee R., Knoxville	8,990	(250,000)	(27.8)	Mar. 10, 1867	135, 132
			195,000	21.7	Mar. 1, 1902	142
722	Cumberland, Celina	7,320	153,000	20.9	Dec. 29, 1926	132
723	French Broad R., Dandridge	4,446	(155,000)	(34.9)	May 21, 1901	135, 132
724	Clinch R., Clinton	3,090	74,300	25	1862	142
725	Holston R., Rogersville	3,060	(200,000)	(65.4)	Mar. 10, 1867	132, 135
			70,900	23.2	Jan. 29, 1918	132
726	Little Tennessee R., McGhee	2,470	(137,000)	(55.4)	Mar. 1867	132, 135
			118,000	47.8	Apr. 2, 1920	142
727	Hiwassee R., Charleston	2,297	70,000	30.5	Mar. 13, 1886	132
			53,500	23.2		142
728	Caney Fk., Silver Point	2,100	178,000	84.8	Mar. 23, 1929	196
729	Hatchie R., Stanton	1,940	59,000	30.4	Jan. 22, 1935	132
730	Obion R., Obion	1,880	99,500	53.0	Jan. 24, 1937	132
731	Little Tennessee R., Calderwood	1,870	70,000	37.4		142
732	French Broad R., Newport	1,860	(160,000)	(86.0)	Feb. 28, 1902	132, 135
			62,200	33.4	Apr. 8, 1903	142
733	Caney Fork, Rock Island	1,640	210,000	128	Mar. 23, 1929	142
734	Duck R., Columbia	1,210	(54,000)	(44.6)	Mar. 30, 1902	132, 135
			43,800	36.2	Mar. 25, 1929	142
735	Hiwassee R., Reliance	1,180	55,100	46.7	Nov. 1906	69
736	Nolichucky R., Greenville	1,140	73,500	64.5		142
737	Elk R., Fayetteville	857	45,600	53.3		142
738	Nolichucky R., Embreeville	795	42,100	53.0	Mar. 26, 1935	132
739	Emery R., Harriman	793	151,000	190	Mar. 23, 1929	196
740	Emery R., Oakdale	764	(150,000)	(196)	Mar. 3, 1929	135
			67,700	88.6	Jan. 2, 1937	132
741	Watauga R., Elizabethton	692	(86,000)	(124)	Feb. 28, 1902	132, 135
			40,000	57.8	July 16, 1916	132
742	Little Tennessee R., Judson	675	57,500	85.2	Dec. 1901	69
743	Collins R., McMinnville	624	75,300	121	Mar. 23, 1929	142
744	Stones R., Smyrna	552	45,000	81.5	Mar. 23, 1929	132
745	Obey R., Byrdstown	452	35,000	77.4	June 29, 1928	142
746	Buffalo R., Flatwoods	439	34,800	79.4		142
747	Little Pigeon R., Sevierville	353	32,000	90.7	June 29, 1928	132
748	New R., New River	312	70,000	224	Mar. 23, 1929	142, 196
749	Piney R., Spring City	97	16,500	170		142
750	Big Rock Cr., near Verona	48.7	26,400	540	June 18, 1939	185
751	Daddy Cr., Grassy Cove	46	14,600	315	Mar. 23, 1929	142
752	Robertson Fork, E. of Lynnville	12.5	6,100	490	June 18, 1939	185
753	Big Rock Cr., above Lewisburg	12.0	9,700	810	June 18, 1939	185
754	Fountain Cr., Culleoka	10.7	7,300	680	June 18, 1939	185

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
755	Belfast Cr., above Farmington	10.2	4,000	390	June 18, 1939	185
756	Fountain Cr., S. Fk., below Campbells Sta.	8.4	6,100	730	June 18, 1939	185
757	Globe Cr., E. Fk., Mackenzie School	6.6	16,300	2,470	June 18, 1939	185
758	Mooresville Cr., near Mooresville	4.2	6,900	1,630	June 18, 1939	185
759	Bear Cr., near Mooresville	3.2	3,300	1,030	June 18, 1939	185
760	Little R., E. Fk., Pigeon	0.4	4,500	11,200	Aug. 30, 1940	189
761	Murchison Farm, Jackson	0.17	323	1,900	Apr. 1918	116
762						
763						
764						
765						
766						
	Texas					
767	Rio Grande, near Del Rio	123,318	604,590	4.9	Sept. 1, 1932	126
768	Colorado R., Austin	26,350	481,000	18.3	June 15, 1935	150
769	San Juan R., Sta. Rosalia	13,000	335,000	26		135
770	Colorado R., near Stacy	11,660	356,000	30.5	Sept. 18, 1936	150
771	Little R., Cameron	7,034	647,000	92.0	Sept. 10, 1921	150
772	Concho R., near Paint Rock	5,257	301,000	57.3	Sept. 17, 1936	150
773	Little R., near Little River	5,240	331,000	63.2	Sept. 10, 1921	150
774	Concho R., near San Angelo	4,217	246,000	58.3	Aug. 6, 1906	150
775	Devils R., near Del Rio	4,060	597,000	147	Sept. 1, 1932	150
776	Llano R., near Castell	3,514	388,000	110	June 14, 1935	150
777	Frio R., near Derby	3,493	230,000	65.8	July 4, 1932	150
778	San Jacinto R., Huffman	2,791	253,000	90.6	Nov. 1940	188
779	Devils R., near Juno	2,733	370,000	135	Sept. 1932	129
780	Nueces R., near Uvalde	1,930	616,000	319	June 14, 1935	150
781	San Jacinto R., near Humble	1,811	187,000	103	Nov. 1940	188
782	Llano R., near Junction	1,762	319,000	181	June 14, 1935	150
783	Lozier Cr., near Langtry	1,728	197,000	114	Sept. 4, 1935	150
784	North Concho R., San Angelo	1,675	184,000	110	Sept. 17, 1936	150
785	Pecan Bayou, near Brownwood	1,614	235,000	146	July 3, 1932	150
786	Pedernales R., near Spicewood	1,294	155,000	120	May 28, 1929	150
787	San Marcos R., Ottine	1,249	202,000	162	May 29, 1929	150
788	Guadalupe R., near Comfort	916	182,000	199	July 1, 1932	150
789	W. Nueces R., near Cline	880	536,000	609	June 14, 1935	150
790	Frio R., near Uvalde	840	148,000	176	July 3, 1932	150
791	San Jacinto, Conroe	832	110,000	132	Nov. 1940	188
792	Nueces R., Laguna	764	213,000	279	June 14, 1935	150
793	Dry Devils R., near mouth	748	129,000	172	Sept. 1, 1932	150
794	Jim Ned Cr., near Brownwood	668	187,000	280	July 3, 1932	150
795	Guadalupe R., Kerrville	570	196,000	344	July 1, 1932	129
796	S. Llano R., near Telegraph	540	160,000	296	June 14, 1935	150
797	Sycamore Cr., near Del Rio	524	215,000	410	June 14, 1935	150
798	Sandies Cr., near Westhoff	493	92,700	188	July 2, 1936	150
799	Frio R., Concan	485	162,000	334	July 1, 1932	131
800	South Concho R., Christoval	434	80,100	185	Sept. 17, 1936	150
801	San Gabriel R., Georgetown	431	160,000	371	Sept. 10, 1921	67

TABLE 1—*Continued*

Unusual Flood Peaks, United States and Foreign Rivers

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
802	Blanco R., near San Marcos	429	139,000	324	May 28, 1929	150
803	W. Nueces R., near Brackettville	402	580,000	1,440	June 14, 1935	150
804	Hondo Cr., near Hondo	400	74,800	187	July 2, 1932	150
805	Blanco R., Wimberley	378	113,000	299	May 28, 1929	150
806	Teneha Cr., near Joaquin	374	117,000	313	July 24, 1933	150
807	Aquilla Cr., near Gholson	372	84,500	227	Sept. 27, 1936	150
808	Frio R., Rio Frio	371	128,000	345	July 1, 1932	150
809	Plum Cr., near Lulina	356	78,500	220	July 1, 1936	150
810	Onion Cr., near Delvalle	337	138,000	409	Sept. 10, 1921	150
811	Guadalupe R., near Ingram	336	206,000	613	July 1, 1932	150
812	James R., near Mason	336	85,900	256	July 2, 1932	150
813	Sabinal R., Sabinal	258	71,700	278	July 1, 1932	150
814	Pinto Cr., near Del Rio	229	54,650	239	Aug. 31, 1932	150
815	Paint Cr., near Telegraph	218	69,300	318	June 14, 1935	150
816	Seco Cr., near D'Hanis	153	230,000	1,500	May 31, 1935	150
817	Onion Cr., near Buda	151	53,200	352	May 28, 1929	150
818	Salado Cr., Salado	148	143,000	966	Sept. 10, 1921	67
819	Dry Frio R., near Reagan Wells	120	64,700	539	June 14, 1935	150
820	Copperas Cr., near Roosevelt	118	98,900	838	Sept. 15–16, 1936	150
821	Barton Cr., near Riley	114	39,400	346	May 28, 1929	150
822	Johnson Cr., near Ingram	111	138,000	1,240	July 2, 1932	150
823	N. Fk., Guadalupe R., near Hunt	110	108,000	982	July 1, 1932	150
824	Terrett Draw, near Ft. McKavett	103	35,800	348	Sept. 16, 1936	150
825	Sandies Cr., near Dewitt	95	54,300	572	July 1, 1936	150
826	Blanco R., near Blanco	92.2	43,500	472	May 28, 1929	150
827	San Antonio R., below San Pedro Creek	85.0	42,427	499	Sept. 10, 1921	150
828	W. Fk., Copperas Cr., near Roosevelt	81	50,400	622	Sept. 16, 1936	150
829	Pecan Cr., near San Angelo	81	30,500	377	Sept. 15, 1936	150
830	Childress Cr., near China Springs	79	47,000	595	Sept. 26–27, 1936	150
831	E. Fk., Frio R., near Leakey	75.0	89,500	1,190	July 1, 1932	150
832	Brushy Cr., Round Rock	74.7	34,500	462	Sept. 10, 1921	67
833	Hamilton Cr., near Marble Falls	67	29,100	435	Sept. 15, 1936	150
834	S. Fk., Guadalupe R., Victoria	65.3	84,300	1,290	July 1, 1932	150
835	San Felipe Cr., Del Rio	62.0	45,000	726	June 1935	126
836	E. Fk., James R., Old Knoxville	60.8	105,000	1,730	July 1, 1932	150
837	Flat Fork Cr., near Center	58.0	42,200	728	July 24, 1933	150
838	N. Fk. of Medina R., Lima	54.0	40,200	744	July 1, 1932	150
839	Grape Cr., near Carlsbad	53	31,800	600	Sept. 17, 1936	150
840	San Pedro Cr., below Apache Creek	46.5	32,443	698	Sept. 9, 1921	150
841	Sabinal R., Vanderpool	45.7	52,300	1,140	July 2, 1932	150
842	San Antonio, San Antonio	34.3	23,700	691	Sept. 1921	67
843	E. Fk., Terrett Draw, below Coal Kiln Draw	33	18,700	567	Sept. 16, 1936	150
844	E. Fk., Grape Cr., near Carlsbad	32	23,500	734	Sept. 17, 1936	150
845	O'Neil Cr., near Leesville	30	30,000	1,000	July 1, 1936	150
846	Olmos Cr., San Antonio	26.4	28,000	1,060	Sept. 9, 1921	150
847	Boggs Cr., near Pueblo	26	15,100	582		71

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
848	Apache Cr., San Antonio	23.8	22,600	948	Sept. , 1921	67
849	Atascosa R., near Benton City	21.3	25,900	1,220	June 22, 1924	150
850	Martinez Cr., San Antonio	19.6	23,900	1,220		71
851	E. Fk., Terrett Cr., above Coal Kiln Draw	19	12,100	637	Sept. 16, 1936	150
852	Alazan Cr., San Antonio	17.1	33,400	1,950	Sept. 1921	165
853	W. Fk., Grape Cr., near Carlsbad	17	14,200	836	Sept. 17, 1936	150
854	Dry Cr., near San Angelo	14	24,600	1,760	Sept. 17, 1936	150
855	Bunton Branch, near Kyle	4.1	13,800	3,370	June 30, 1936	150
856	Sevenmile Draw, Ames	2.4	5,140	2,140	Sept. 26, 1936	150
857	Red Bank Cr., near San Angelo	0.76	2,490	3,280	Sept. 17, 1936	150
858						
859						
860						
861						
862						
	Utah					
863	Green R., Blake	38,200	67,200	1.76	May 1897	120
864	Virgin R., Virgin City	1,010	12,000	11.9	1912	72
865	Weber R., Oakley	163	4,080	25		69
866	Farmington Canyon, Farmington	7.0	2,450	350	Aug. 1923	74
867	North Canyon, near Centerville	4.0	1,800	450	Aug. 1923	74
868	China Wash, near Hurricane	1.1	550	500	Aug. 1916	74
869						
870						
871						
872						
873						
	Vermont					
874	Connecticut R., White River Junction	4,068	136,000	33.4	Nov. 4, 1927	136
875	Winooski R., Essex Junction	1,070	116,000	108	Nov. 4, 1927	145
876	White R., West Hartford	690	120,000	174	Nov. 4, 1927	136
876a	Winooski R., Montpelier	433	57,000	132	Nov. 3, 1927	136
877						
878						
879						
880						
881						
	Virginia					
882	James R., near Richmond	6,757	158,000	23.4	Mar. 1936	172
883	Staunton R., Randolph	3,080	75,000	24.4	Dec. 1901	120
884	Dan R., South Boston	2,730	81,000	29.6	Aug. 16, 1940	135
885	New R., Radford	2,725	174,000	63.8	1900	21a
886	James R., Buchanan	2,084	92,200	44.2	Mar. 27, 1913	172
887	Shenandoah R., S. Fk., near Front Royal	1,638	113,000	69	Mar. 18, 1936	172
888	Rappahannock, near Fredericksburg	1,599	66,000	41.3	May 13, 1924	154

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
889	James R., N. Fk., Glasgow	831	37,200	44.8	1896	71
890	Roanoke R., Roanoke	388	28,000	72.0	Aug. 14, 1940	135
891	Craig Cr., Parr	331	21,500	65	Jan. 23, 1935	176
892	Powell R., Pennington	304	28,900	95.2		142
893	Blackwater R., near Union Hall	208	19,700	95	Aug. 14, 1940	135
894						
895						
896						
897						
898						
	Washington					
899	Columbia, Grand Coulee	70,000	492,000	7.03		150
900	Clark Fk., Newport	24,200	217,000	8.98	June 1894	69
901	Yakima R., Kiona	5,520	63,500	11.5	Nov. 1906	72
902	Yakima R., Union Gap	3,550	63,900	18.0	Nov. 1906	99
903	Yakima R., Umtanum	1,620	41,000	25.3	Nov. 1906	178
904	Cowlitz R., Mossy Rock	1,170	50,900	43.5	Nov. 1906	72
905	Yakima R., Cle Elum	500	25,600	51.2	Nov. 1906	178
906	Cle Elum Lake, Roslyn	202	18,700	93	Nov. 1906	178
907	Baker R., near Anderson Cr.	184	36,800	200	Dec. 1917	119
908	Cedar R., Landsberg	136	13,600	100	Nov. 19, 1911	178
909	Wynoochee R., near Montesano	105	25,000	238	Feb. 11, 1924	99
910						
911						
912						
913						
914						
	West Virginia					
915	Ohio R., Parkersburg	37,950	650,000	17.1	Mar. 30, 1913	165
916	Ohio R., Wheeling	23,800	507,000	21.3	Feb. 1884	99
917	Kanawha, Kanawha Falls	8,376	270,000	32.2	Sept. 14, 1878	172
918	Potomac R., Shepherdstown	5,936	335,000	56.5	Mar. 19, 1936	135
919	Shenandoah R., Millville	3,040	150,000	49.4	Mar. 1936	172
920	Monongahela R., Hoult	2,430	91,500	38	Jan. 1919	187
921	Potomac R., S. Br., near Springfield	1,471	143,000	97.2	Mar. 1936	172
922	Cheat R., Morgantown	1,380	160,000	116	July 1888	187
923	Greenbriar R., Alderson	1,344	62,600	46.5	Mar. 1913	69
924	Tygart R., Fetterman	1,304	74,300	57	July 1912	187
925	Big Sandy R., Tug Fk., Kermit	1,185	(70,000)	(59.1)	Mar. 29, 1913	132
926	Elk R., Queen Shoals	1,145	91,300	79.7	July 5, 1932	132
927	Cheat R., Rowlesburg	972	65,200	67	Feb. 1932	187
928	Monongahela, W. Fk., Enterprise	759	(70,000)	(91.5)	July 10, 1888	132
929	Cheat R., near Parsons	719	85,000	118	July 1888	187
930	Gauley R., Summerville	686	92,000	134	July 4, 1932	144
931	Cacapon R., near Great Cacapon	670	103,000	154	Mar. 1936	172
932	Middle Island Cr., Little	458	(45,000)	(98.3)	Aug. 1875	132
933	Coal R., Ashford	393	40,700	104	Aug. 9, 1916	132
934	Potomac R., S. Fk. of S. Br., near Moorefield	271	43,000	159	Mar. 1936	99

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
935	Shavers Fork, Parsons	230	25,000	109	July 1907	187
936	Big Sandy Cr., Rockville	200	30,000	150	July 1907	187
936a	Shavers Fk., Cheat Bridge	57.5	11,000	191	July 1896	187
937	Elkhorn Cr , Keystone	44	60,000	1,360	June 1901	34
938						
939						
940						
941						
942						
	Wisconsin					
943	Mississippi R., Prescott	45,000	134,000	2.98	Apr. 30, 1881	132
944	Wisconsin R., Muscoda	10,300	80,800	7.84	Sept. 16, 1938	132
945	Wisconsin R., Kilburn	8,000	80,000	10.0		90
946	Chippewa R., Eau Claire	6,740	60,700	9.0	June 1905	1
947	Wisconsin R., Necedah	5,800	93,400	16.1	June 1905	120
948	Chippewa R., Chippewa Falls	5,600	78,000	13.9	Mar. 27, 1920	132
949	Wisconsin R., near Merrill	2,780	45,000	16.2	July 24, 1912	132
950	Black R., Neillsville	675	23,100	34.2		69
951						
952						
953						
954						
955						
	Wyoming					
956	Big Horn R., Hardin	20,700	40,800	1.97	1908	72
957	Big Horn, Thermopolis	8,080	29,800	3.7	July 24, 1923	125
958	Powder R., Arvada	6,050	95,000	15.7	Sept. 1923	112
959	Salt Cr., below Reservoir	794	48,400	61	Sept. 1923	112
960	Salt Cr., Sec. 36 T.41N, R.79W.	520	32,000	61.5	Sept. 27, 1923	196
961	Laramie Reservoir Outlet, Laramie	72.0	6,980	97	Mar. 1913	38
962						
963						
964						
965						
966						
	Foreign					
967	Amazon R. at mouth, Brazil	2,368,000	7,110,000†	3.0		96
968	Amazon R., Obidos, Brazil	1,945,000	6,810,000†	3.5		96
969	Yangtze Kiang R., China	1,100,000	3,000,000†	2.73		92
970	Ganges R., India	367,970	1,800,000	4.9		93
971	Irrawaddy R., India	149,800	1,900,000†	12.7		93
972	Rhine, German-Dutch border	86,620	459,000	5.3		181
973	Rhine, Emmerich, Germany	62,000	425,000	6.9		165
974	Fitzroy R., Australia	58,000	613,000	10.6	Feb. 1896	85
975	Danube, Vienna, Austria	39,400	495,000	12.5	1501	107
976	Cagayan R., Luzon, Philippine Islands	4,100	980,000	239	Dec. 4, 1936	135
977	San Juan R., China, Mexico	3,360	250,000	74		138
978	Chagres R., near Gatun, Panama	1,320	124,000	93.9	Dec. 28, 1909	32

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No.	Stream and Location	Drainage Area, sq mi	Flood, sec-ft	Flood, sec-ft per sq mi	Date of Flood	Authority
979	Musi R., Hyderabad, India	862	425,000†	493	1908	107
980	Ardeche R., at Junction with Rhone, France	831	318,000	382	1827	63
981	Towbrapoorny R., India	587	191,000	324		107
982	Santa Catarine R., Monterey, Mexico	544	235,000	432	Aug. 27, 1909	91
983	Krishna R., India	345	118,000	343		93
984	Irrity R., India	336	150,000	446		107
985	Ardeche R., Aubenas, France	178	123,500	694	1890	63
986	Orba R., at reservoir, Italy	58	80,000	1,380	Aug. 1935	130
987	Tansa R., India	52.5	35,000	667		93
988	Orba R., Valle Orbicella, Italy	42	54,600	1,300	Aug. 1935	128
989	Wailua, near Lihue, Kauai, Hawaii	23	45,000	1,950	Jan. 1921	78
990	Elbe R., headwaters, Germany	20	35,000	1,750	July 9, 1927	165
991	Orba R., Martina, Italy	18.3	34,200	1,870	Aug. 1935	128
992	Orbicella R., Italy	10	20,100	2,010	Aug. 1935	128
993	Kaneohe, Oahu, Hawaii	5.3	11,000	2,070	Jan. 1921	78
994	Kaukonahua, Upper Dam, Oahu, Hawaii	4.5	7,220	1,600	Jan. 1921	78
995	Nuuanu, Res. No. 4, Oahu, Hawaii	1.5	2,400	1,600	Feb. 1907	78
996	Manoa, E. Br., Oahu, Hawaii	1.1	3,090	2,810	Jan. 1921	78
997	Manoa, W. Br., Oahu, Hawaii	1.0	3,250	3,250	Jan. 1921	78

LIST OF AUTHORITIES

REFERRED TO IN TABLE 1

1 to 65: *Trans. Am. Soc. Civil Engrs.*, 1914, p. 662. Authorities listed by the late Emil Kuichling
66 to 120: *Trans. Am. Soc. Civil Engrs.*, 1926, p. 1030. Authorities listed by C. S. Jarvis.
122: *Eng. News-Record*, Vol. 115, p. 162.
125: *U. S. Geol. Survey Water-Supply Paper* 806.
126: *Intern. Boundary Commission Ann. Repts.* (United States and Mexico).
128: *Eng. News-Record*, Vol. 117, p. 474.
129: *Eng. News-Record*, Vol. 117, p. 655.
130: *Eng. News-Record*, Vol. 115, p. 610.
131: *U. S. Geol. Survey Water-Supply Paper* 748.
132: *U. S. Geol. Survey Water-Supply Papers*, miscellaneous.
133: *U. S. Geol. Survey Water-Supply Paper* 773E.
134: The Floods of March 1936 in Pennsylvania; State Publication, 1936, table of flood records.
135: Unofficial or unpublished hydrologic data.
136: *J. Boston Soc. Civil Engrs.*, Oct. 1936, p. 291.
138: *Mexican Commission Irrig. Repts.*
139: *Eng. News-Record*, Vol. 117, p. 243.
140: *Am. Civil Engrs. Handbook*, 1930 Ed., p. 1418.
141: *H. Doc.* No. 149, 72nd Cong., 1st Sess., p. 19.
142: *Tennessee Geol. Survey Bull.* 40, 1931.
144: *U. S. Geol. Survey Water-Supply Paper* 758.

LIST OF AUTHORITIES—Continued

REFERRED TO IN TABLE 1

145: *U. S. Geol. Survey Water-Supply Paper* 759.
146: *Eng. News-Record,* Vol. 116, p. 836.
147: *U. S. Geol. Survey Water-Supply Paper* 783.
148: Dept. Water and Power, Los Angeles.
149: *Military Eng.,* May–June 1937, p. 168.
150: *Civil Eng.,* July 1937, p. 493.
152: *U. S. Geol. Survey Water-Supply Paper* 796B.
153: *U. S. Geol. Survey Water-Supply Paper* 798.
154: *U. S. Geol. Survey Water-Supply Paper* 781.
155: *U. S. Geol. Survey Water-Supply Paper* 662.
156: *U. S. Geol. Survey Water-Supply Paper* 697.
157: *Eng. Experiment Sta. Bull.* 296, Univ. Illinois.
158: *J. New England Water Works Assoc.,* Sept. 1937.
160: *Eng. News-Record,* Vol. 114, p. 178.
161: *Eng. News-Record,* Vol. 120, p. 227.
162: *Eng. News-Record,* Vol. 120, p. 349.
164: Los Angeles County Flood Control District.
165: *Report on Flood Control in Germany,* by W. W. Wanamaker, published by U. S. Eng. School, Fort Belvoir, Va., Feb. 1, 1938.
166: *U. S. Geol. Survey Water-Supply Paper* 799.
167: *U. S. Geol. Survey Water-Supply Paper* 809.
169: *Trans. Am. Geophys. Union,* July 1937, Part 2, p. 409.
170: *Eng. News-Record,* Vol. 95, p. 280.
171: *U. S. Geol. Survey Water-Supply Paper* 811.
172: *U. S. Geol. Survey Water-Supply Paper* 800.
173: *U. S. Geol. Survey Water-Supply Paper* 813.
174: *U. S. Geol. Survey Water-Supply Paper* 785.
175: *U. S. Geol. Survey Water-Supply Paper* 561.
176: *U. S. Geol. Survey Water-Supply Paper* 782.
177: *U. S. Geol. Survey Water-Supply Paper* 814.
178: *U. S. Geol. Survey Water-Supply Paper* 812.
180: *Western Constr. News,* Sept. 1938, p. 328.
181: *Military Engr.,* May–June 1938, p. 167.
182: *Civil Eng.,* June 1938, p. 406.
183: *Eng. News-Record,* Vol. 121, p. 428.
184: *J. Boston Soc. Civil Engrs.,* Oct. 1938, p. 499.
185: T. V. A. Rept. on Storm of June 18, 1939, in Vicinity of Lewisburg, Tenn.
186: *U. S. Geol. Survey Water-Supply Paper* 842.
187: *Flood Discharge Records Relating to Pennsylvania Streams,* Div. Hydrography, Dept. Forests and Waters, Commonwealth of Pennsylvania.
188: Board of Water Engineers, Austin, Texas.
189: *North Carolina Civil Engr.,* August 1941, p. 6.
191: *U. S. Geol. Survey Water-Supply Paper* 872.
192: *Biennial Report on Hydrologic Data,* Los Angeles County Flood Control District, June 30, 1939.
194: *Floods of August 1940 in Tennessee River Basin,* T. V. A. mimeographed report.
195: U. S. Geol. Survey press release, Nov. 1941 (preliminary data).
196: *U. S. Geol. Survey Water-Supply Paper* 847, "Maximum Discharges at Stream-Measurement Stations through Sept. 1938."
197: U. S. Engr. Office records (published in *Low Dams,* item 370*b*).
198: Unpublished and Preliminary Data from Los Angeles District Office of the United States Geological Survey.
200: *Flood of March 2, 1938,* by M. F. Burke, Hydraulic Engineer, Los Angeles County Flood Control District, Appendix Table No. 7.
201: "Report on Tehachapi Flood Flows, Storm of September 1932" (unpublished,) Los Angeles County Flood Control District.

a different drainage area of the same characteristics and under the same meteorological conditions, such as another point on the same river or another similar river.

It is known that, with other conditions the same, the greater the drainage area, the smaller the flood per square mile of area, the most commonly used equation being

$$Q = C^1 A^n \qquad [2]$$

or its equivalent

$$q = C^1 A^{n-1} \qquad [2a]$$

where Q = the flood in sec-ft;
q = the flood in sec-ft per sq mi;
A = the drainage area in sq mi;
n = an exponent which is less than unity;
C^1 = a coefficient depending upon the characteristic of the drainage basin.

Values assigned to n by various authorities have ranged from 0.3 to 0.8. A value of $n = 0.8$ was found by Fuller (Art. 65, Ref. 45) with the data available in 1913. Later Creager (Art. 65, Ref. 39), with the data available in 1926, found $n = 0.5$, and Myer (Art. 65, Ref. 31, p. 994) also made use of $n = 0.5$ in his much-quoted "Myer's Equation." Hazen (Art. 65, Ref. 32) found $n = 0.8$ for Atlantic Coast streams based on mean floods.

In a study of floods in Colorado, Wyoming, and Arizona, Hodges [2] found n to vary with different localities, ranging from $n = 0.3$ to $n = 0.69$ with an average of $n = 0.45$. However, the number of observations were said not to be sufficient for accurate determinations for the individual areas.

A reference to Fig. 1 indicates that, with the exception of a few floods of rather remarkable magnitude, an enveloping curve can be drawn which indicates an approximate variation of flood peak per square mile with drainage area. Such a curve has been drawn in Fig. 1.

This indicates that the value of n is not constant but takes the approximate form of

$$n = \frac{n^1}{A^k} \qquad [3]$$

and the equation of the enveloping curve [3] the form of

$$Q = 46C'A^{(0.894A^{-0.048})} \qquad [4]$$

or its equivalent

$$q = 46C'A^{(0.894A^{-0.048})-1} \qquad [4a]$$

In Fig. 1 the enveloping curve for $C = 100$ follows the general trend of all maximum floods with the exception of floods resulting from two storms, the

[2] *Eng. News-Record*, August 10, 1933, p. 171.

[3] Eqs. 4 and 4a are not "flood formulae" in the sense usually known (see Art. 9).

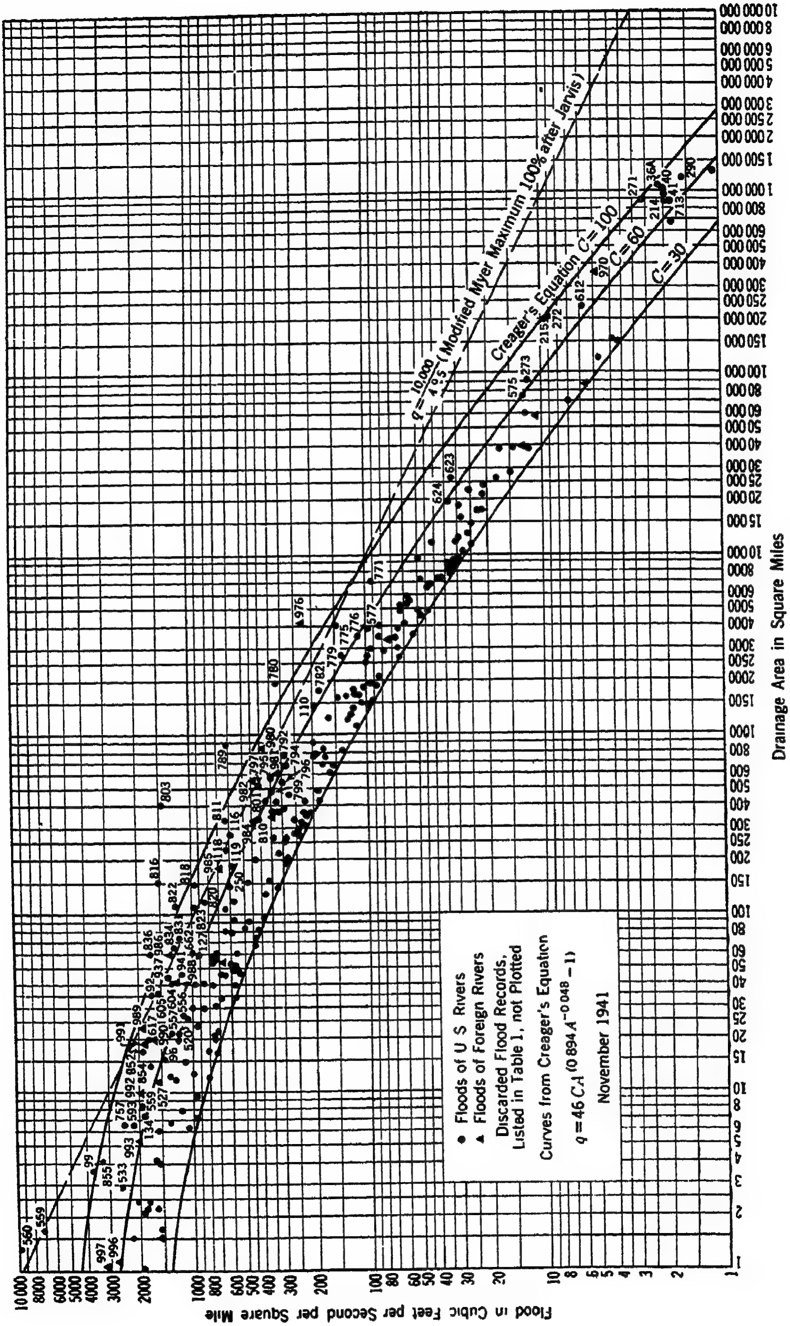


FIG. 1. Record of unusual flood discharges.

1940 storm in North Carolina (floods 559, 560, and 760[4]) and the May–June 1935 Texas storm (floods 780, 789, 803, 816). There was also an unofficial record of an extraordinary flood in the Philippines (976).

These floods are inexplicable. They were estimated by the United States Geological Survey, the Texas records being again reviewed at the request of the author. Therefore, it must be assumed that these floods were the result of meteorological conditions so far different from those for usual great storms as to put the floods in a class by themselves and hence they would not affect the general trend of the enveloping curve.

The "Modified Myer's Maximum $100\%_C$" developed by Jarvis (Art. 65, Ref. 31, p. 994), which is the most used of the equations of the form of Eqs. 2 and 2*a*, has been shown in Fig. 1 solely for the purpose of comparison with the proposed new Eqs. 4 and 4*a*.

The equation of the enveloping curve for districts having smaller flood peaks than that corresponding to $C = 100$ has not yet been found, but it is believed that for the present the use of Eq. 4*a* with C less than 100 is the best we have available. Eq. 4*a* is useful in drawing enveloping curves for plotted floods in limited districts. Such curves can be drawn easily by multiplying the ordinates of $C = 100$ in Fig. 1 by a constant to obtain a curve for any desired value of C.

It may be demonstrated that, in consideration of the laws of variations of average storm rainfall, lag, and average topographic features with area, the form of Eq. 4 is more logical than that of Eq. 2. In other words, studies which have been made by the writer indicate that the enveloping curve should be flatter than the average for the smaller areas and steeper for the larger areas when plotted to logarithmic scale.

The first step in the use of Table 1 in flood studies comprises an investigation of the climatological features of the surrounding section of the country in order to determine the limits within which such features may be considered approximately the same as those for the drainage area in question.

A search is then made for record floods throughout that section of the country and these are plotted in the form used in Fig. 1. An enveloping curve, plotted from Eq. 4*a* or reducing the curve for $C = 100$ in Fig. 1 as explained, would then indicate the flood characteristics of the maximum flood-producing streams in that section. One must be sure that the section considered is large enough, has had its share of floods, and has been sufficiently gaged that its records are indicative. Otherwise a larger section must be considered, even though it extends into greater flood-producing sections. As the ultimate limit, Fig. 1 could be used.

Having obtained the enveloping curve, the next step is to determine whether the flood peak producing characteristics of the stream in question are as great as, or greater than, the maximum characteristics in that district, as indicated by that curve. Without evidence to the contrary it should be considered at least as great, recognizing always the possibility that it might be greater.

[4] 11,200 sec-ft per sq mi from 0.4 sq mi is off the figure.

For the purpose of obtaining this evidence, the following methods of study have been used.

4. Comparison of Physical Characteristics. A knowledge of the physical factors affecting the magnitude of floods is essential in the investigation of flood flows, particularly if comparisons are desired with other streams on which the flood tendencies are known. Aside from differences in area of the watershed, two streams may have materially different flood tendencies, accounted for by a difference in the characteristics of the watershed. The flood coefficient C of Eqs. 2 and 4, the use of which is made necessary by such differences in characteristics, depends on many factors, the chief of which are:

(1) Storm rainfall characteristics:
  (*a*) Type of storms.
  (*b*) Characteristics of storms, including distribution of intensity with time.
  (*c*) Effect of proximity of oceans.
  (*d*) Trend of great storms.
  (*e*) Effect of mountain ranges.
(2) The storage capacity of the watershed, or its ability to retain water temporarily above or below ground, and to regulate the runoff:
  (*a*) Storage in artificial reservoirs.
  (*b*) Storage in lakes and swamps.
  (*c*) Storage below ground surface.
  (*d*) Storage or retardation at ground surface.
  (*e*) Valley storage.
(3) Slopes of drainage areas.
(4) Shape of the drainage area.
(5) Direction of the general trend of the river in comparison with prevailing directional movement of rainfall centers.
(6) Capacity of barriers in the stream to release waters suddenly. (Storm rainfall is discussed in Part II of this chapter.)

Storage, of whatever nature, tends to reduce the peak of floods. The effect on the flood peak of artificial reservoirs must be considered as a separate problem as described in Part II of this chapter. The effect of lakes, swamps, ground storage, surface storage, and valley storage must be evaluated according to judgment and experience.

The magnitude of floods is always less on rivers draining deep sandy areas in which the storage below the ground is considerable. If such areas are large and extend to the higher elevations of the watershed, the effect on floods may be considerable.

Storage above the ground is affected by the nature of the vegetation, the shape and slope of the drainage area, and the characteristics of the river bed and banks. It is evident that those characteristics which will permit rapid runoff of the precipitation to the site of the dam will result in large floods. Rocky slopes, devoid of vegetation, are conducive to quick discharge. Con

versely, areas covered with dense vegetation will prove effective in holding back the water and smoothing out the peak of the flood. (See Ref. 42, Art. 65.) Heavy grasses and underbrush are particularly effective in this respect. Forests also retard the melting of snow. Practically no water, at the peak of the precipitation, is held back by adherence to leaves and branches of forests above the ground surface. For this reason many engineers in this country are of the opinion that it is the removal of the dense underbrush and surface humus rather than of the large trees that has increased flood tendencies in deforested districts.

On the other hand, a commission of engineers studying floods in Germany has concluded that forests tend to mitigate the smaller and middle floods [5] and that only in long and continued rainfall is this influence lost.

Valley storage is the amount of water required to fill the river channel and valley to high-water elevation during floods. This sometimes has a tremendous influence on peak discharge. The peak of the great flood of 1869 on the Black River at Watertown, N. Y., was exceeded in magnitude at Lyons Falls, on the same river where the drainage area was less than half, owing to the valley storage on the flats between the two places.[6]

Frequent restrictions in the river valley will tend to increase valley storage.

Steep slopes will produce rapid runoff. Therefore floods from mountainous districts are relatively severe.

In rivers having tributaries extending in the shape of a fan from a given point, and of approximately the same size, the peak of the flood from each of the tributaries is likely to reach the main stream and the dam at approximately the same time, resulting in relatively large floods. Conversely, when the catchment area is relatively narrow, with tributaries of different sizes discharging into the main stream at regular intervals, the peak of the runoff from the tributary areas will reach the dam at different times, resulting in relatively small floods. A large number of tributaries is also productive of rapid runoff.

The creation of extremely long reservoirs materially decreases the time of runoff.

If a rainfall center passes up the river, the runoff during the first of the rainfall period reaches the point in question at a different time than that from the last part; but if it passes down the river at about the same rate as the water flows, all of the runoff tends to reach the point in question at the same time with a resultant higher peak. Therefore streams that lie generally parallel to the direction of movement of rainfall centers and flow in an opposite direction have smaller flood probabilities than those which flow with the storms.

The capacity of the catchment area to release stored water suddenly may be indicated by:

(1) The frequency and magnitude of ice and log jams, with consequent danger of release of impounded waters at or near the peak of the flood.

[5] W. W. Wanamaker, *Floods in Germany*, U. S. Army Eng. School, February 1, 1938.

[6] Cullings and Hazen, "Report on Control of Floods," *Eng. News-Record*, Vol. 83, p. 28, for Lyons Falls. New York Development Assn., Watertown, N. Y., 1928, for Watertown.

(2) The presence of other dams of questionable strength or insufficient spillway capacity, impounding large volumes of water. A number of well-designed dams have failed on account of the failure of defective dams above, with a resultant enormous increase in the runoff due to the sudden release of stored waters.

(3) Temporary partial blocking of the flow of the stream, due to the lodgment of debris against submerged bridges, and subsequent failure of the bridges, with a release of the impounded waters at the critical time.

(4) Storage in the form of snow, which may be suddenly released by a record precipitation accompanied by a rise in temperature.

The first three of these items are indeterminates in theoretical flood studies. If danger from them exists and can be computed, the resultant flow must be added to the calculated flood.

Melting of snow has an enormous effect on floods. In some floods the maximum recorded discharge has been due almost entirely to this cause. Therefore, other things being equal, areas draining snow-covered lands would be expected to have larger floods than those which do not. The influence of snow-melt is discussed in Art. 40.

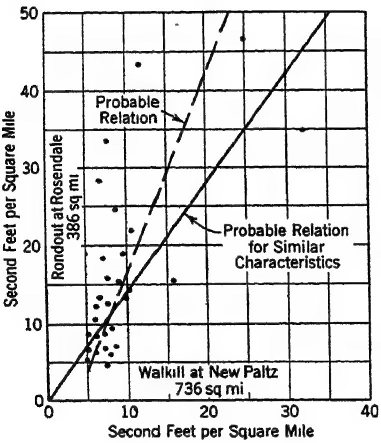


Fig. 2. Comparison of flood flows, Wallkill River and Rondout Creek.

It is probable that, in general, the area of the watershed, the maximum rate and duration of rainfall and melting snow, the steepness of the slopes, the geologic formation, the slope of the drainage area and arrangement of tributaries, and the nature of the vegetation will, in the order given, have the most influence on the flood tendencies of the stream.

The flood characteristics of two adjacent streams may be compared as illustrated in Fig. 2, which indicates the relation between simultaneous floods on the Wallkill River and Rondout Creek, N. Y., adjacent watersheds.

From Eq. 4, the flood runoff per square mile from the 736 sq mi of the Wallkill watershed would ordinarily be about 71 per cent of that from the 386 sq mi of the Rondout watershed, other conditions being the same. This theoretical relation is indicated by the full line in Fig. 2. Actually, however, the probable relation is a line drawn through the plotted floods, as indicated by the dash line, showing that the Rondout Creek has greater relative flood tendencies than the Wallkill. The Wallkill watershed has flatter slopes and a greater extent of swamps. Small floods on the Wallkill are, however, relatively larger than on the Rondout, which is accounted for by the fact that a small artificial reservoir on the Rondout just above Rosendale smooths out the smaller floods;

but the reservoir is too small to have any appreciable effect on the large floods.

Relations of this kind furnish a means by which the flood characteristics of a stream may be obtained approximately with relatively short periods of runoff records, if long-term records are available on a neighboring stream.

Where flow records on a stream are not available, it is necessary to determine the flood characteristics of surrounding streams and apply those characteristics, as judgment dictates, to the stream in question, making allowance, if possible, for differences in those features which affect flood runoff.

Where sufficient data and time are available, a rational study of this type can be made if the variables are not too numerous. This was done by G. T. McCarthy for a study of Connecticut River floods,[7] and a very reasonable relation was found between the flood characteristics, i.e., rate of peak discharge, duration of flood, and peaking time on the one hand and three topographic features, i.e., area, slope, and stream pattern on the other hand.

It will be noted that other features herein mentioned as affecting flood flows were not a part of this study. Possibly those features were fairly uniform for the Connecticut River watershed.

5. Flood Frequency Studies. The probable frequency of a flood of a given magnitude may be determined on a mathematical basis by the laws of probability, provided that the records of river discharge upon which the study is based are *truly representative of average conditions.*

It will be shown in Art. 6 that few stream flow records, particularly on small streams, are of sufficient length to be truly representative of average conditions and therefore are valueless for predicting the frequency of floods beyond a few hundred years. Therefore the use of probability methods, for the design of spillways according to a 1,000- or a 10,000-year criterion, is extremely hazardous without supporting evidence and is no longer standard practice. However, such methods will be described here in a modified form [8] because they have the following important uses:

(1) Comparisons of the coefficient of variation as explained in Art. 7.

(2) Special studies involving small frequencies, such as the probability of overtopping cofferdams or frequency of damage to unimportant properties by flooding.

(3) The economic justification of flood control.

The method involves the following steps:

(1) The plotting of existing flood records in the form of a frequency probability curve, as in Fig. 3.

(2) The extrapolation of this curve to intervals of time, I, greater than the length of the records. For example, in Fig. 3, a flood of 75,000 second-feet may be expected on the average of once in $I = 300$ years.

[7] Appendix I to Connecticut River Survey Report, H. Doc. No. 455, 75th Cong., 2nd Sess.

[8] For full description see *Engineering for Masonry Dams*, 2nd Edition, by W. P. Creager; John Wiley & Sons, New York, 1929, superseded by this volume.

The law of probabilities, as applied to a study of flood frequencies, may be made by either of the following methods:

(*a*) *Basic Stage Method.* A consideration of all floods that exceeded a given basic stage during the period of record.

(*b*) *Yearly Flood Method.* The use of only the maximum flood during each year of record.

The basic stage method was advanced in 1926 by Creager (Ref. 39, Art. 65) as being the more accurate, although it involves the use of a larger number of factors. However, the results from the two methods are not materially different and, in view of our more recent knowledge of the inaccuracy of probability curves for extended extrapolation, the yearly method is used herein.

An estimate of the probable frequency of 24-hour flood peaks, based on discharge records of the Hocking River for the years 1916 to 1937 inclusive is described below and is shown in Table 2.

The maximum 24-hour average yearly peaks, Q, are tabulated in Col. 3 in their order of size. The number of times, n, that each flood was equaled or exceeded is shown in Col. 4.

According to the law of probabilities, the probable percentage of years in which a flood equal or greater than a given discharge, Q, will occur may be obtained from the following equation:

$$p = \frac{100n}{y} \qquad [5]^{9}$$

where p = the probable percentage of future years in which a flood equal or greater than a discharge, Q, will occur;

n = the number of years during the period of record that a flood, Q, was equaled or exceeded, as shown by Col. 2.

y = the total number of years of record (in this case 22 years).

This equation gives the values of p in Col. 5, which indicates that 4.5 per cent of future years may expect a flood equal or greater than 30,900 sec-ft.

Values of peak discharge (24-hour average flow used in this case) from Col. 1 are plotted as ordinates, and percentages from Col. 3 as abscissae, on probability paper,[10] as indicated in Fig. 3. A curve drawn through the plotted points may be extended to obtain an extrapolated value of the flood, Q, which may be expected to be equaled or exceeded, on an average, in p per cent of future years. In this example a 24-hour flood of 53,000 sec-ft may be

[9] The equation most commonly used is

$$p = \frac{100(n - 0.5)}{y} \qquad [5a]$$

but the writer has never been entirely satisfied with its theoretical derivation, and because of the necessity for conservatism in connection with floods he advocates the use of Eq. 5, which gives a greater frequency of floods.

[10] Probability paper is published by Codex Book Co., Norwood, Mass., with either arithmetic or logarithmic ordinates. Use whichever gives straightest curve. Originally devised by Allen Hazen, *Trans. Am. Soc. Civil Engrs.*, December 1914, p. 1539.

TABLE 2

CALCULATION FOR PROBABILITY PLOTTING OF FLOOD DISCHARGES OF HOCKING RIVER AT ATHENS, OHIO, 1916–1937

1	2	3	4	5	6	7	8	9
Year	Maximum 24-hr Average Yearly Peak, Q, (sec-ft)	Same as Col. 2, but Arranged in Order of Magnitude	Number of Times, n, Peak Was Equaled or Exceeded	Percentage of Years p	Future Flood Frequency, I (years)	Col. 3 in Terms of Mean Flood	Variation from Mean V	V^2
1916	10,000	6,200	22	100.0	1.00	0.436	−0.564	0.318
1917	9,960	6,700	21	95.3	1.05	0.471	−0.529	0.280
1918	12,800	6,800	20	90.8	1.10	0.479	−0.521	0.271
1919	16,100	9,960	19	86.3	1.16	0.701	−0.299	0.089
1920	16,500	10,000	18	81.7	1.22	0.704	−0.296	0.088
1921	24,000	10,100	17	77.2	1.29	0.710	−0.290	0.084
1922	28,700	10,300	16	72.7	1.37	0.724	−0.276	0.076
1923	10,300	11,200	15	68.1	1.47	0.787	−0.213	0.045
1924	14,600	11,300	14	63.6	1.57	0.794	−0.206	0.042
1925	6,700	11,700	13	59.0	1.69	0.822	−0.178	0.032
1926	11,200	11,900	12	54.5	1.83	0.837	−0.163	0.027
1927	14,000	12,800	11	50.0	2.00	0.900	−0.100	0.010
1928	13,700	13,700	10	45.4	2.20	0.963	−0.037	0.001
1929	11,300	14,000	9	40.9	2.44	0.984	−0.016	0.000
1930	11,700	14,600	8	36.3	2.75	1.026	+0.026	0.001
1931	10,000	16,100	7	31.8	3.14	1.131	+0.131	0.017
1932	6,200	16,500	6	27.3	3.67	1.160	+0.160	0.026
1933	18,200	17,000	5	22.7	4.40	1.195	+0.195	0.038
1934	6,800	18,200	4	18.2	5.50	1.279	+0.279	0.078
1935	17,000	24,000	3	13.6	7.33	1.687	+0.687	0.472
1936	11,900	28,700	2	9.1	11.00	2.017	+1.017	1.034
1937	30,900	30,900	1	4.5	22.00	2.172	+1.172	1.374
		312,660						4.403

Mean Flood $= \dfrac{312{,}660}{22} = 14{,}200$ $\qquad \Sigma V^2 = 4.403 \qquad CV = \sqrt{\dfrac{4.403}{21}} = 0.457$

expected to be equaled or exceeded during 1 per cent of future years. The approximate momentary peak can be obtained from Eq. 1.

The interval in years, between floods equaling or exceeding a given flood, Q, is

$$I = \frac{100}{p} \qquad [6]$$

This equation is used to plot the scale at the top of Fig. 3 and to compute the values of Col. 6 of Table 2. For this example a flood of 53,000 sec-ft may be expected to be equaled or exceeded, on an average, once in 100 years.

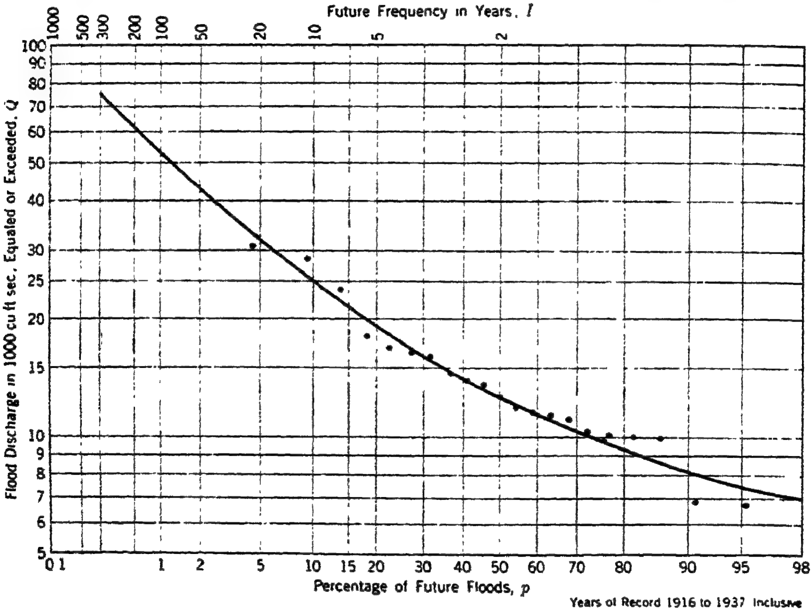


FIG. 3. Probability curve of Hocking River at Athens, Ohio.

The drawing of the probability curve through and beyond the plotted points can be and formerly was made by mathematical methods involving Pearson's and other functions. (See Ref. 32, Art. 65.) However, it is now considered that the curve may be drawn by eye well within the accuracy of the probability method.

The "coefficient of variation" or steepness of the probability curve affords a means of comparing the flood-producing characteristics of one area with another, as explained in Art. 7. The coefficient of variation may be obtained as follows:[11]

Referring to Table 2, the summation of Col. 3, divided by the years of record, y, equals the mean flood, or 14,200 sec-ft.

[11] H. ALDEN FOSTER, "Theoretical Efficiency Curves and Their Application to Engineering Problems," *Trans. Am. Soc. Civil Engrs.*, 1924.

Col. 7 tabulates the floods of Col. 3 in terms of the mean flood.

Col. 8 shows the variation, V, of Col. 7 from the mean, i.e., Col. 7 minus 1.0.

The coefficient of variation, CV, is given by the following equation:

$$CV = \sqrt{\frac{\Sigma V^2}{y-1}} = \sqrt{\frac{4.403}{22-1}} = 0.457 \qquad [7]$$

where ΣV^2 is the summation of Col. 9.

6. Inherent Defects in Flood Frequency Studies. About 1914 the theory of probabilities was applied to flood studies; i.e., curves were derived indicating by past records on a stream the frequency with which, during a long period, a given flood should be expected. Notwithstanding the fact that periods of record sometimes did not exceed 20 years and very seldom exceeded 30 or 40 years, these probability curves were extrapolated to estimate the flood which would be expected, during long periods, once in 1,000, 5,000, or 10,000 years. Then, according to the judgment of the engineer, the 1,000-, 5,000-, or 10,000-year flood was selected for the design capacity of the spillway.

So seriously was this method of estimating floods taken that numerous articles appeared in the technical publications defining more exact methods for such extrapolation by use of Pearson's and other functions, the writer included chapters on the methods in two books, other books included a description of the method, and the late Allen Hazen devoted an entire book to the subject.

Recently, however, it has been proved by advanced studies and a greater accumulation of data, that the probability method is entirely inadequate. Data on floods of many years ago and gagings of more recent floods have proved conclusively that there must be a combination of meteorological conditions which give rise to storms of a special class which occurs so infrequently that the resulting floods seldom appear on the published records of a given river. These storms and their resulting floods seem to be in a different class from ordinary floods and follow some law of their own.

Thus floods have occurred on rivers which, based upon probability studies of prior records of considerable length, would have a frequency not of the usually adopted 1,000 to 10,000 years but a frequency of once in millions and even billions of years.

Hazen (Art. 65, Ref. 32, pp. 53, 87) recognized this peculiarity of floods but, because of lack of verifying data, he disregarded this possibility in his analysis of floods and believed that it should be considered an indication of the necessity of using the most conservative methods. But, since that time, the phenomenon has been repeated so often as to change the possibility to practically a certainty.

As one of the many recent examples, the Republican River in Nebraska experienced a flood in 1935 which was ten times as large as had ever occurred on that river during 39 prior years of record. The probability method would never in the world have indicated the possibility of such a storm. However, more recent methods of prognostication would indicate that possibility very

clearly. The probability method has therefore been abandoned except for special cases where the frequency of the smaller floods is to be studied, as mentioned in Art. 5.

7. Comparison of Coefficients of Variation. To assist in the comparison of the flood-producing characteristics of a stream with those of record-breaking streams, the coefficient of variation, one of the functions of the flood probability theory described in Art. 5, may be used. However, in such a study, the out-of-line or special-class floods mentioned in Art. 6 should not be included, as such a flood may have occurred on one river and not the other.

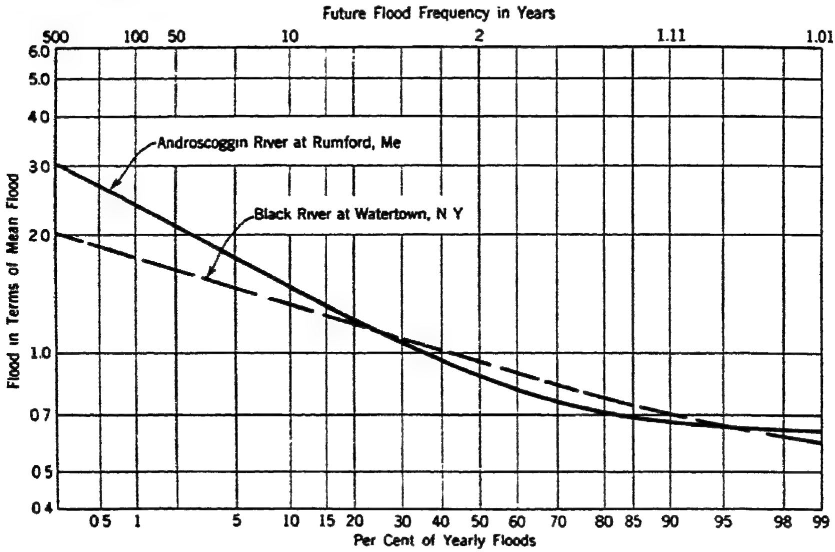


FIG. 4.

In Fig. 4 is shown probability curves of the Androscoggin and Black Rivers. However, instead of using the actual Q as ordinates, as in Col. 3 of Table 2, the floods in terms of the mean flood, as in Col. 7, were used. Other data are given in Table 3.

TABLE 3

COMPARISON OF PROBABILITY CURVES OF TWO RIVERS

	Androscoggin	Black
Drainage area, sq mi	2,090	1,870
Mean yearly flood, sec-ft	23,366	21,960
Coefficient C for mean flood, in Eq. 4	4.9	4.8
Coefficient of variation (Art. 5)	0.380	0.259

It will be noted that, although of about the same area and having about the same mean flood coefficient, the greater divergence of the Androscoggin, as shown by Fig. 4, and its greater coefficient of variation indicate its greater flood characteristics.

8. Physical Indications of Past Floods. Authentic Federal and State Government records of high water, extending over long periods, may be obtained for many streams.[12] Such records are also often available from mill operators and officials of municipalities. In general, however, the elevation of record high water must be determined from the observations and traditions of residents and from physical indications on the banks of the stream.

Observations and traditions of residents should be regarded with caution. Individual reports of untrained observers are subject to great error and, strange to say, are often of doubtful veracity, as the desire to report high water a little higher than that reported by a neighbor is often, among certain classes, greater than the love of truth. Unfortunately, also, reports are sometimes biased by a desire to give an impression of great or small floods, whichever, in the opinion of the observer, will better serve his interests. However, credence may be given when a number of observations closely agree, and are referred to definite objects, such as sills of doors and windows or nails driven for reference.

Confirmation may be obtained from the elevation of deposited brush, logs, or alluvial matter, scars from floating logs on banks and large trees, and whatever other indications of high water may be discovered. High water, in an alluvial valley which has been formed from the sediment deposited from floods, is, of course, always higher than the surface of such deposits.

The elevation of record high water having been fixed, there are four methods by which an estimate of the corresponding discharge can be made. (See also Art. 65, Ref. 33.)

(1) The head on a dam which existed at the time of such high water can be determined and from this the discharge over that structure can be computed by one of the well-known weir formulas.[13]

(2) In unusual cases where the loss of head at contracted openings between bridge abutments has been observed, the approximate discharge can be computed from the expected loss at such openings.[14]

(3) If a considerable stretch of straight river having a nearly uniform cross-section and slope is available, an estimate of the discharge can be made by use of Kutter's formula for the flow in open channels, particularly if accurate current-meter measurements of smaller floods have been made in order to determine the coefficient of roughness of the channel.

(4) By the projection of a rating curve to the elevation of high water.[15] This method, however, is only available as a rough indication of the corre-

[12] U. S. Geological Survey, U. S. Weather Bureau, Engineer Department, U. S. Army, etc.

[13] See R. E. HORTON, *U. S. Geol. Survey Water-Supply Paper* 200.

[14] See Report of the Chief Engineer, Miami Conservancy District, Vol. 1, March 1916, p. 63.

[15] A rating curve shows the relation between gage height and discharge as indicated by discharge measurements.

sponding flood, unless the cross-section of the river is particularly regular and the discharge measurements used in plotting the curve cover a range including floods amounting to considerable proportions.

Slope-area methods of determining discharge, mentioned in Item 3, are subject to three major uncertainties:

(a) Scour of the river bed during rising floods and subsequent refill as the flood recedes.
(b) Nonuniform rate of rise of water surface in the length of the stretch of river being used for the gagings.
(c) The choice of Kutter's coefficient, n.

(a) The beds of silt-laden rivers flowing on flat alluvial deposits may scour greatly because of the high velocities during floods and refill subsequently as the flood recedes. For this reason it is necessary, for accurate slope-area gagings, to know the cross-sections, or at least obtain a few soundings, at the peak of the flood. Measurements of the cross-section after the passing of the flood are worthless in some cases.

Sometimes deposits of immovable material located by borings, or other deposits which the geologist may classify as of ancient origin, will indicate the maximum possible scour which occurs during recent floods. In other cases, bridge piers which are known to have stood a flood without the aid of foundation piles, will indicate by the elevation of their foundations the maximum scour which could have occurred.

(b) The slope of the water surface is steeper for the rising flood than it is during highest water or during the receding flood. Thus the maximum discharge may not occur when the water surface is highest. According to Sir William Willocks,[16] the Tigris River had the following discharges:

15 foot gage height when rising	180,000 sec-ft
20 " " " at peak	120,000 " "
15 " " " when falling	90,000 " "

This is probably a very extreme case. Similar cases have been observed by the Mississippi River Commission.

(c) The best indication of the value of Kutter's n, for use in slope-area gagings of a large flood, is obtained from slope-area measurements in conjunction with some later smaller flood which has been gaged by another method.

Values of n vary for natural channels generally between 0.025 and 0.035. However, floods overflow upon a different character of surface which sometimes may be considerably rougher and may even be wooded. For such surfaces the values of n frequently reach 0.040 to 0.060 and occasionally may be in excess of 0.100.

Photographs of river types with recommendations for values of n will be found in "Flow of Water in Drainage Channels," by C. E. Ramser, Tech. Bull. 129, November 1929, U. S. Dept. Agr. See also "Some Better Kutter's Formula Coefficients," by R. E. Horton, *Eng. News*, February 24, 1916, p 373

[16] *Eng. Record*, July 4, 1914, p. 16.

9. Flood Formulas. Most flood formulas for maximum flood discharge are empirical to the extent that they take into consideration only a small number of the many factors affecting flood runoff. Hayford [17] has shown that there may be no less than 23 factors affecting the magnitude of floods.

Most of the formulas embody a coefficient which is left either to the judgment or imagination of the user or is stated to lie within certain rather wide limits. Some are applicable only for the region for which they were derived.

In some publications the constants applicable to flood-flow formulas are given in the form of charts which purport to show their variation throughout the entire United States. In the use of such charts it must be remembered that the constants vary widely in areas much smaller than can be shown on the charts.

Results of the application of different formulas to the same drainage area differ widely. A description and analysis of the many flood formulas which have been proposed are contained in Ref. 26 of Art. 65. No recommendations are given with regard to the relative accuracy of the many formulas described therein. On the other hand, it is stated that "at best a general formula is only a temporary substitute for observed or logically derived flood information...."

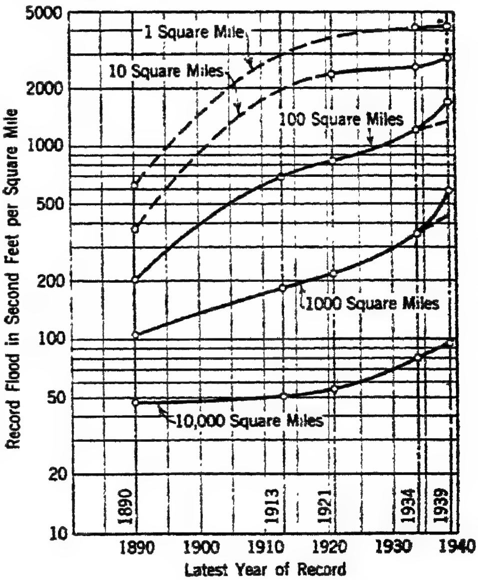


FIG. 5. Trend of record floods.

For these reasons, flood formulas are not included in this discussion. The foregoing Eqs. 4 and 4*a* are intended to give only the variation with area and not actual discharge, since the value of C for a given district is not indicated.

10. Possible Future Peaks. In making use of records of maximum recorded floods on rivers in a given district to estimate the expected peak discharge at a given place, it must be remembered that what has occurred in the past must surely be exceeded in the future. Allowance must be made for this fact; and no definite recommendations can be given, as opinions in this respect differ materially.

For a fuller discussion of this subject, the reader is referred to Ref. 38 of Art. 65, from which Fig. 5 is taken. This figure shows the approximate magnitude of the greatest recorded floods of past United States records prior to

[17] HAYFORD and FOLSE, "A New Method of Estimating Stream Flow," *Carnegie Inst.*, Washington, Pub. 400, 1929.

several different years. It indicates how, as time passes and more data are available, the record flood from any size drainage area continues to increase. For instance, the greatest known flood from 100 sq mi in 1890 was 200 sec-ft per sq mi, whereas in 1939 it was 1900 sec-ft or 8.5 times as great. It is expected that a similar plotting of record storms would show like characteristics.

II. FLOOD HYDROGRAPHS[18]

By Gail A. Hathaway and A. L. Cochran[19]

A. BASIC HYDROLOGIC ANALYSIS

11. General. It is the primary purpose herein to suggest a procedure for estimating hypothetical hydrographs to represent "maximum probable" or limiting volumes and concentration of runoff, for use in determining design capacities of spillways for dams or other important hydraulic structures in which a high degree of security against failure is mandatory.

The characteristics of natural drainage basins and meteorological influences affecting runoff are too complex and variable to be evaluated accurately by mechanical computation procedures alone. Even if it were practicable to analyze the runoff phenomena with scientific accuracy, the limitations of basic data would generally preclude such studies. The reliability of estimates of runoff rates and quantities is dependent as much upon the soundness of judgment used in the interpretation of data as in the detail of the computation procedures applied. A thorough study of the characteristics and meteorological influences affecting runoff from a specific basin constitutes an essential basis for sound judgment.

The accuracy of a hypothetical hydrograph designed to represent runoff under specified conditions and the reliability of a rational analysis of recorded hydrographs are dependent upon the ability to evaluate the predominating influences of the following factors on runoff:

Rainfall
- *a.* Intensity, duration, sequence.
- *b.* Areal distribution during successive time intervals.

Infiltration
- *a.* Initial loss, or loss before appreciable runoff begins.
- *b.* Minimum average capacity, or in some cases, the relation of capacity to field-moisture conditions.

[18] An effort has been made to acknowledge definite sources of material used by the writers in the preparation of Arts. 11 to 64. In addition to such direct references, it is desired to give particular credit to the various district and division offices of the U. S. Engineer Department, Corps of Engineers, for the development of methods of hydrologic computation procedures and analyses that have been incorporated with various degrees of modifications in the procedures suggested herein.

[19] Gail A. Hathaway, C.E., Member, American Society of Civil Engineers, Head Engineer, Office, Chief of Engineers, U. S. Army, Washington, D. C.

A. L. Cochran, C.E., Associate Member, American Society of Civil Engineers, Engineer, Office, Chief of Engineers, U. S. Army, Washington, D. C.

Regimen of Runoff

a. Effects of basin configuration and arrangement of tributaries.
b. Effects of natural storage:
   1. In tributaries, lakes, swamps, etc.
   2. In principal stream channels and valleys.
c. Effects of artificial structures:
   1. Reservoirs.
   2. Channel improvements.
   3. Land-use practices.
d. Effects of slopes:
   1. In principal stream channels and flood plains.
   2. In drainage areas tributary to principal runoff channels.
e. Effects of land coverage:
   1. Forested areas.
   2. Cultivated areas.
   3. Pasture lands and barren areas.
f. Ability of subsurface soil to transmit infiltrated water to surface channels within the period required for direct runoff to pass through the channel storage phase of runoff.

B. Rainfall Analysis

12. Mass Rainfall Curves. One of the most convenient methods of estimating the intensity and chronology of rainfall at various points in a basin involves the construction of mass rainfall curves similar to those shown in Fig. 7. The mass curve is primarily a means of recording the conclusions reached by correlating such miscellaneous data as may be available regarding rainfall intensity and time of concurrence. The following routine is ordinarily used in its construction.

(*a*) A preliminary total-storm isohyetal map similar to Fig. 6 is prepared from published precipitation records to delineate the general rainfall pattern. If the storm is characterized by two or more distinct intervals of heavy rainfall, isohyetal maps are prepared for each principal rainfall period.

(*b*) Mass curves for recording precipitation stations are plotted on a composite sheet, in order to facilitate a study of progressive variations in rainfall intensities within the storm area. Mass curves for the recording stations nearest the principal rainfall zones may indicate intensities that are representative of the type of storm involved and may aid in interpolating mass rainfall curves for intermediate nonrecording precipitation stations.

(*c*) Data for nonrecording precipitation stations, including notes regarding times of beginning and ending of rainfall, cloudiness, direction and velocity of the wind, and any other pertinent notes are transcribed from original observers' records to convenient forms. Some of these records are available in publications of the U. S. Weather Bureau, but in important studies the published records should be supplemented from the following sources:

1. The original unpublished notes of the Weather Bureau observers, on file in the central Weather Bureau Office, Washington, D. C., and in the

Climatological Section Centers of the Weather Bureau located in numerous cities of the United States.

2. Published and unpublished records of state, municipal, and other public water-supply and conservation agencies.
3. Files of private organizations interested in water power, water supply, etc.
4. Files of local newspapers, and records maintained by residents in the storm area.

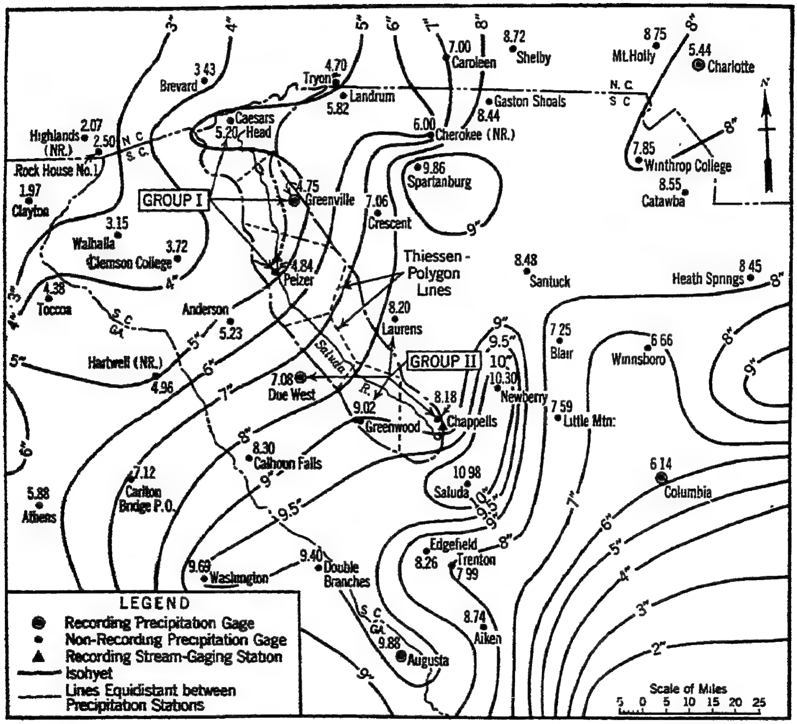


FIG. 6. Saluda River Basin above Chappells, S. C. Isohyetal map, Sept. 30 to Oct. 2. 1929.

(*d*) A group of four to six neighboring stations that are located in regions of reasonably similar topography, and that appear to have been subjected to similar meteorological conditions during the storm, is selected. The accumulative rainfall at successive recording times is plotted for each of these stations on a transparent form to permit comparison with groups of curves plotted on other sheets. Similar plottings are made for other groups of stations surrounding the first group.

(*e*) The mass rainfall curves are completed by interpolating the curves between established points in such a manner as to reflect reasonable consistency with the period of rainfall at neighboring stations, with frontal and convective

activity as determined from meteorological analyses, and any additional data that are available in the specific instance. Such data are seldom entirely consistent; consequently, the most logical interpretations must be decided upon as the study progresses.

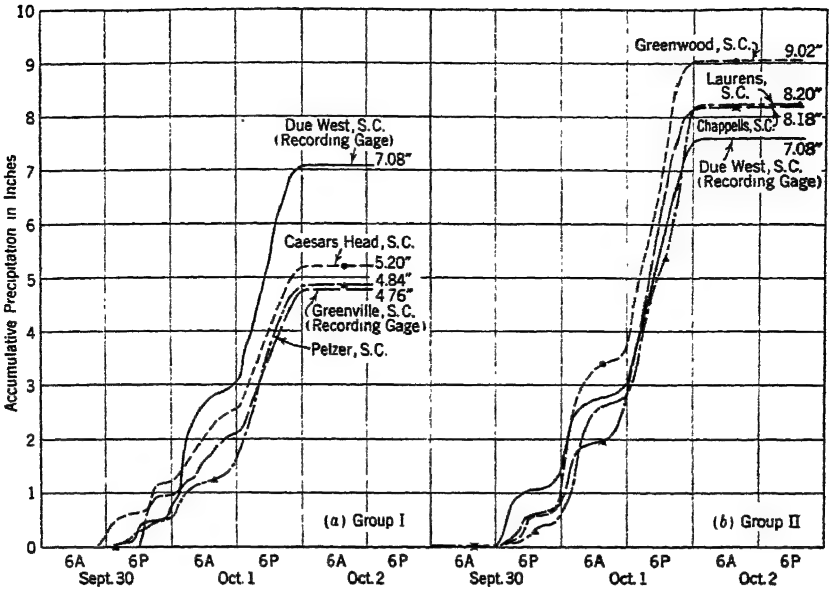


FIG. 7. Mass rainfall curves for Sept. 30 to Oct. 2, 1929.

13. Hyetographs. The term "hyetograph" is used herein to refer to the graphical representation of average rainfall and rainfall-excess rates, or volumes, over specified areas during successive units of time during a storm. Hyetographs are convenient in connection with the analysis of flood hydrographs. Examples are shown in Figs. 8, 9, and 17.

C. INFILTRATION

14. General. It has been demonstrated that the capacity of a given soil to absorb rainfall applied continuously at an excessive rate rapidly decreases until a fairly definite minimum rate of infiltration is reached, usually within a period of a few hours (Art. 65, Refs. 1, 2, 3, 5). The order of decrease in infiltration capacity and the minimum rate attained is dependent primarily upon the size of soil pores within the zone of aeration and the conditions affecting the rate of removal of capillary water from the zone of aeration.

The infiltration theory, with certain approximations, offers a practical means of estimating the volume of surface runoff from intense rainfall in humid regions. However, in applying the method to natural drainage basins, the following facts must be considered.

(*a*) The infiltration capacity of a given soil at the beginning of a period of rainfall is related to antecedent field moisture and the physical condition of the soil. Accordingly, the infiltration capacity for the same soil varies appreciably.

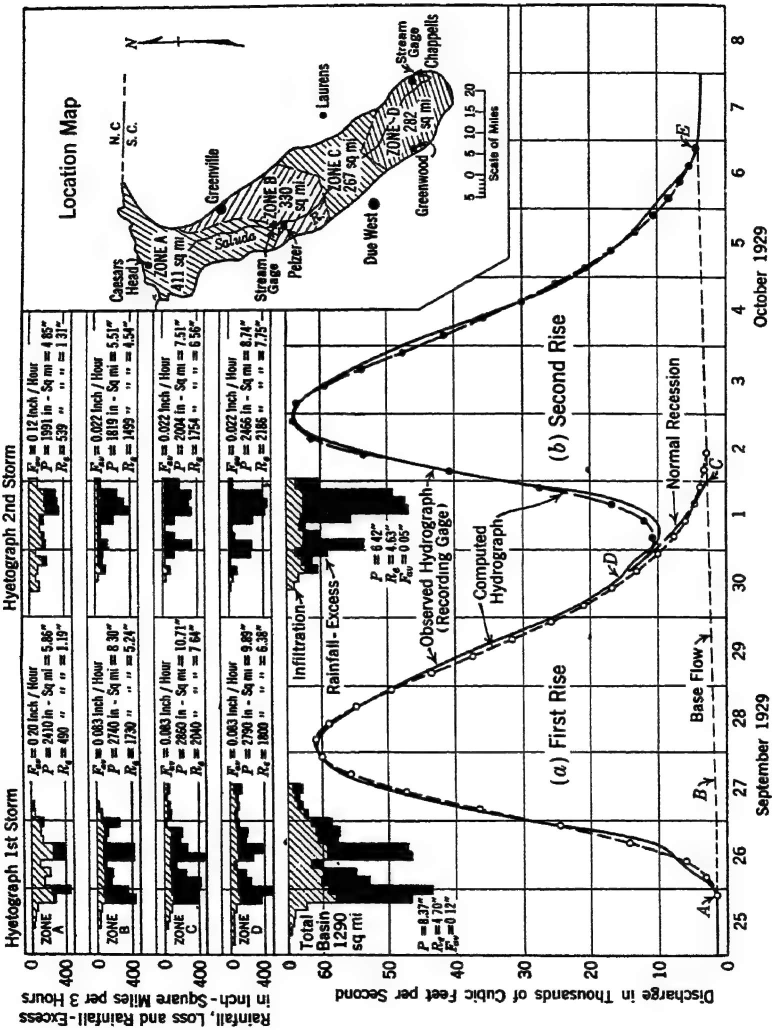


Fig. 8. Hydrograph of Saluda River at Chappells, S. C.

(*b*) Inasmuch as the infiltration capacity of a soil is normally highest at the beginning of rainfall, whereas rainfall frequently begins at relatively moderate rates, a substantial period may elapse before the rainfall intensity exceeds the prevailing infiltration capacity. Actually, studies have indicated that a fairly definite quantity of water-loss by infiltration is required to satisfy initial field moisture deficiencies before runoff will occur, the amount of loss depending

upon antecedent conditions. A practical application of the infiltration theory in estimating runoff from moderate rainfall intensities ordinarily includes an allowance for "initial losses" corresponding to various antecedent field moisture conditions.

(*c*) Rainfall over relatively large drainage basins frequently does not cover the entire basin during all periods of precipitation with intensities exceeding infiltration capacities. Therefore a rational application of the infiltration theory to large drainage areas requires consideration of rainfall intensities in various portions of the basin in order to determine, at least approximately, the area covered by effective runoff-producing rainfall.

(*d*) During actual flood-producing storms, the rainfall in various parts of a drainage basin is more or less intermittent. Such interruptions in precipitation modify the order of decrease in infiltration capacity that might otherwise be expected according to the infiltration theory.

15. Initial Loss. Initial loss is defined as the maximum amount of precipitation that can occur under specific conditions without producing runoff (Art. 65, Ref. 6). Initial loss values for basins in humid areas of the United States may range from a minimum of a few tenths of an inch during relatively wet seasons to approximately 2 inches during dry summer and fall months. The initial loss for conditions usually preceding major floods in humid regions normally ranges from about 0.2 to 0.5 inch and is relatively small in comparison with the flood runoff volume. Consequently, in computing infiltration indices from data for major floods, allowances for initial loss may be neglected or estimated approximately, without introducing important errors in the results.

Franklin F. Snyder (Art. 65, Ref. 6) has suggested a procedure for estimating initial loss values corresponding to field moisture conditions as reflected by the ground-water discharge. The procedure is recommended for consideration in connection with problems requiring estimates of runoff from moderate storms.

16. Infiltration Index. In view of the approximations involved in infiltration estimates, the average loss rate computed from rainfall-runoff data for natural drainage basins will be referred to herein as "infiltration index" rather than as "infiltration capacity." Infiltration index (*Fav*) is defined as an average rate of loss such that the volume of rainfall in excess of that rate will equal the volume of direct runoff.

The following procedure is recommended for the computation of infiltration indices to be used in estimating the volume of runoff from major storms in large drainage basins:

(*a*) Hydrographs of the major floods of record in the basin are selected for study; the volume of surface runoff is computed by subtracting from the recorded gross volume of runoff the estimated base flow and the runoff from extraneous rainfall (see Table 4, Col. 29, lines 23, 32, and 34).

(*b*) Isohyetal maps similar to Fig. 6 are drawn for each principal rainfall period, and mass rainfall curves (Fig. 7) are constructed.

TABLE 4

Computation of Infiltration Indices for Saluda River Basin Above Chappells, S. C.

Part I. Rainfall Volumes

Storm Period: 1929, Sept. 30–Oct. 2
45 Hours from 10 A.M. Sept. 30 to 7 A.M. Oct. 2

[Ap = Area within station-polygon and basin. Psta = Total rainfall at observation station.
Pav = Average depth rainfall within area Ap. Initial Loss = Infiltration required before rainfall-excess begins.]

Line No.	Drainage Basin	Precipitation Station	Ap in Sq Mi	Psta in Inches	Pav in Inches	Pav/Psta	$Ac = Ap\left(\frac{Pav}{Psta}\right)$	Volume of Rainfall within Area Ap, in In.-sq mi (Equals Ac × 3-Hour Rainfall Increments)																				Total
									Time at End of 3-Hour Periods																			
									1P30	4P30	7P30	10P30	1A1	4A1	7A1	10A1	1P1	4P1	7P1	10P1	1A2	4A2	7A2					
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
1																												
2	Saluda River	Pelzer, S. C.	47	4.8	4.7	0.98	46		0	5	11	6	11	20	3	4	33	49	43	30	7	0	0					222
3	basin above	Greenville, S. C.	112	4.8	4.6	0.96	107		0	0	58	42	8	31	45	36	21	90	79	78	20	0	1					509
4	Pelzer, S. C.,	Caesars Head, S. C.	252	5.2	5.0	0.96	242		99	46	10	131	34	114	99	63	75	182	191	177	39	0	0					1260
5	gaging station																											
6		Sub-Total	411		4.85				99	51	79	179	53	165	147	103	129	321	313	285	66	0	1					1991
7																												
8	Saluda River	Chappells, S. C.	116	8.2	8.7	1.06	123		5	27	41	9	58	87	12	28	139	213	190	163	33	0	0					1005
9	basin between	Greenwood, S. C.	166	9.0	8.8	0.98	162		0	23	65	11	201	188	60	16	146	219	264	236	32	0	0					1461
10	Chappells, S. C.	Laurens, S. C.	151	8.2	7.6	0.93	140		0	11	28	15	64	168	74	15	48	202	129	227	167	0	0					1148
11	and Pelzer, S. C.,	Due West, S. C.	116	7.1	7.4	1.04	121		1	10	46	4	54	171	53	13	85	128	175	92	24	0	0					856
12	gaging stations	Pelzer, S. C.	237	4.8	5.8	1.21	287		0	32	66	40	72	126	17	26	204	304	270	189	43	0	0					1389
13		Greenville, S. C.	93	4.8	4.7	0.98	91		0	0	49	35	6	26	38	31	18	76	67	66	17	0	1					430
14																												
15		Sub-Total	879		7.15				6	103	295	114	455	766	254	129	640	1142	1095	973	316	0	1					6289
16		Grand Total	1290						105	154	374	293	508	931	401	232	769	1463	1408	1258	382	0	2					8280
17																												

TABLE 4—*Continued*

PART II. RAINFALL-EXCESS VOLUMES

Line No.	Drainage Basin	Precipitation Station	Ap in Sq Mi	Initial Loss		Fav		Volume of Rainfall-Excess Within Area Ap, Corresponding to Fav, in Inch-Square Miles																				Total
				Inches	Inch-Sq Mi	In./Hr	Inch-Sq Mi (Per 3 Hrs)	Time at End of 3-Hour Periods																				
									1P30	4P30	7P30	10P30	1A1	4A1	7A1	10A1	1P1	4P1	7P1	10P1	1A2	4A2	7A2					
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
18																												
19	Saluda River basin above Pelzer, S. C., gaging station	Pelzer, S. C.	47	0.5	24	0.125	18		0	0	0	0	0	2	0	0	15	31	25	12	0	0	0					85
20		Greenville, S. C.	112	0.5	56	0.125	42		0	0	2	0	0	0	3	0	0	48	37	36	0	0	0					126
21		Caesars Head, S. C.	252	0.5	126	0.125	94		0	0	0	37	0	20	5	0	0	88	97	83	0	0	0					330
22		Sub-Total	411		206		154		0	0	2	37	0	22	8	0	15	167	159	131	0	0	0					541
23																						Actual surface runoff						539
24																												
25	Saluda River basin between Chappells, S. C., and Pelzer, S. C., gaging stations	Chappells, S. C.	116	0.3	35	0.022	8		0	0	33	1	50	79	4	20	131	205	182	155	25	0	0					885
26		Greenwood, S. C.	166	0.3	50	0.022	11		0	0	38	0	190	177	49	5	135	208	253	225	21	0	0					1301
27		Laurens, S. C.	151	0.3	45	0.022	10		0	0	0	5	54	158	64	5	38	102	119	217	157	0	0					1009
28		Due West, S. C.	116	0.3	35	0.022	8		0	0	22	0	46	103	45	5	77	120	167	84	16	0	0					745
29		Pelzer, S. C.	237	0.3	71	0.022	16		0	0	27	24	56	110	1	10	188	288	254	173	27	0	0					1158
30		Greenville, S. C.	93	0.3	28	0.022	6		0	0	21	29	0	20	32	25	12	70	61	60	11	0	0					341
31		Sub-Total	879		264		61		0	0	141	59	396	707	195	70	581	1083	1036	914	257	0	0					5439
32																						Actual surface runoff						5431
33		Grand Total	1290						0	0	143	96	396	729	203	70	596	1250	1195	1045	257	0	0					5980
34																						Actual surface runoff						5970

TABLE 5

Infiltration Indices

NUMBER OF VALUES OF Fav WITHIN VARIOUS RANGES, AS COMPUTED FROM HYDROLOGIC RECORDS FOR NATURAL DRAINAGE BASINS

Basin	State	Drainage Area in sq mi	January–February Fav, In. per Hr							March–April Fav, In. per Hr							May–June Fav, In. per Hr							July–August Fav, In. per Hr							September–October Fav, In. per Hr							November–December Fav, In. per Hr						
			0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25
1	2	3	4							5							6							7							8							9						
Northeastern U. S. Drainage																																												
Housatonic R.	Conn.	618																																							1		
Quinebaug R.	"	712																																									1
Connecticut R.	Vt.	2,750																																					1				
" "	"	4,040																																					1				
" "	Mass.	7,170																																					1				
" "	"	7,740																																					1				
Millers R.	"	372																																						1			
Swift R.	"	186																																							1		
Tully R.	"	50								1																							1						1				
Ware R.	"	201																																							1		
Westfield R.	"	496																																					1				
Carrabassett R.	Maine	351																																				1					

Dead R.	"	830																																										1
Mattawamkeag R.	"	1,500																																				1				
Ossipee R.	"	455																																								1
E. Br. Penobscot R.	"	1,070																																				1				
Penobscot R.	"	4,690																																					1			
Piscataquis R.	"	286																																					1			
" "	"	1,170																																						1		
Pleasant R.	"	325																																			1					
Saco R.	"	1,300																																								1
Ammonoosuc R.	N. H.	395																													1							1				
Ashuelot R	"	71								1	1																					1						1				
Contoocook R.	"	55																																					1			
" "	"	363																																1								
Merrimack R.	"	935																																				1				
Nubanusit R.	"	54																																								1
Pemigewasset R.	"	599																																			1					
Smith R.	"	79																																								1
Souhegan R.	"	168																																					1			
Sugar R.	"	223								2									1													1							1			
Winnepesaukee R.	"	471																														1							1			
Delaware R.	N. Y.	3,070										1							1													1	3		1	2						
Susquehanna R.	Pa.	7,770																					1						1				1	1	1	1						
Southeastern U. S. Drainage																																										
Saluda R.	S. C.	411							1																								2		1	1						
" "	"	1,290					1													1						1			1			1	1		1							

TABLE 5—*Continued*

INFILTRATION INDICES

NUMBER OF VALUES OF Fav WITHIN VARIOUS RANGES, AS COMPUTED FROM HYDROLOGIC RECORDS FOR NATURAL DRAINAGE BASINS

Basin	State	Drainage Area in sq mi	January–February							March–April							May–June							July–August							September–October							November–December						
			Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr						
			0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25
1	2	3	4							5							6							7							8							9						
Saluda R.	S.C.	1,570				1														1							1		1							1								
" "	"	2,320																		1							1			1			1				1							
" "	"	2,450				1								1																														
Lower Mississippi Valley																																												
Boggy Bayou	La.	108	1	1	1									1		1																									1			
Cypress Bayou	"	60		3								2																												1				
Wallace Lake	"	251	2	1										2																										1				
Arkansas and Red River Basins																																												
Buffalo R.	Ark.	1,118		2	5		1				1	3		2				2	2	1	1																							
Eleven Point R.	Mo.	662		1		1						1	3	1	1	1					1																							
" " "	Ark.	9.5			2								2	1						1																								
Fourche La Fave R.	"	682	2		2							1																																

Lee Cr.	"	430		2							1							1			1																					1	
Litt'e Red R.	"	1,120		4						1	1	4	1					1	3																				2				1
North Fork R.	Mo.	1,150			1	2	1					3		1	2					2	1		1																				
" " "	Ark.	1,590			2	2							1		2					2	1	1																					
Strawberry R.	"	420		2	2	2						1	1	2					1			1								1		1		1								1	
White R.	"	1,270	1								1	3								1	1																		1			
Grand (Neosho) R.	Kans.	3,795										2											1											1	1								
Medicine Lodge R.	"	1,000																			1		3																				
Ninnescah R.	"	2,092																				1	1																				
Verdigris R.	"	2,952										2									3	1										2	1										
Walnut R.	"	1,894																					3												1								
Black R.	Mo.	957			1							3								1																							
Current R.	"	1,805										2																															
Spring R.	"	1,160										2				1						1	2			1								1									
Caney R.	Okla.	750														1				1	1		2																				
" "	"	2,030									1									1	1																						
Cimarron R.	"	3,200																																			1						
" "	"	17,935																																			1						
Deep Fork R.	"	2,300												1		1							2																				
Illinois R.	"	933		2	1	1								1				1		1									1														
" "	"	1,603										1										1																			1		
Poteau R.	"	1,015		1								1																															
" "	"	1,240		1									2																														
Salt Fk., Arkansas R.	"	1,020														1						1	3																				
Stillwater Cr.	"	165														2							3							1							1						
Wolf Cr.	"	1,100														1						1																					
" "	"	1,660														1							1																				

TABLE 5—*Continued*

Infiltration Indices

NUMBER OF VALUES OF Fav WITHIN VARIOUS RANGES, AS COMPUTED FROM HYDROLOGIC RECORDS FOR NATURAL DRAINAGE BASINS

Basin	State	Drainage Area in sq mi	January–February							March–April							May–June							July–August							September–October							November–December						
			Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr							Fav, In. per Hr						
			0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25	0 to 0.02	0.02 to 0.05	0.05 to 0.10	0.10 to 0.15	0.15 to 0.20	0.20 to 0.25	Over 0.25
1	2	3	4							5							6							7							8							9						
Ohio River Basin																																												
Embarrass R.	Ill.	1,540																				1	1														1							
Wabash R.	Ind.	3,830																					1							3							2							
East Fk., White R.	"	4,940				1																																						
Big So. Fk., Cumberland R.	Ky.	1,260	2	2																																			2					
Cumberland R.	"	2,010	1	3																																			2					
Muskingum R.	Ohio	5,980																			1	1								3														
Conemaugh R.	Pa.	1,357								1									2															1										
Redbank Cr.	"	528								1									1																				1					
Caney Fork R.	Tenn.	1,695		1	1						1		1					1																					1					
" " "	"	2,130	1	3	1						2			1			1																					1		1				
Collins R.	"	624		4																																			2					
French Broad R.	"	4,450									1	1	1	1									1				1							1			1							

Harpeth R.	"	404		4																																		1	1				
" "	"	687	1	2	1																																	1	1				
" "	"	695		3	4						2	1																										1	1				
New R.	"	383	2	2																																	1	1					
Obey R.	"	452	2	2	3								1						1																			2	1				
Red R.	"	678	1	1	3						1		1					1																				2	1				
Stone R.	"	550	3	3	1						2								1																			4					
" "	"	865		2		2																																					
North Pacific Drainage																																											
Coyote Cr.	Oreg.	132																																					1				
Long Tom R.	"	81		1							1							1																			1						
Row R.	"	211								1	1																					1						1	1				
North Santiam R.	"	470	1	2	1						1																											1					
Coast Fk., Willamette R.	"	69		1						1	1																											1					
" " " "	"	520	2		1					1																												1					
South Pacific Drainage																																											
Los Angeles R.	Calif.	155														1																											
Tujunga Wash	"	81														1																											

(*c*) The drainage area is subdivided by a series of Thiessen polygons defining areas that are nearest the various precipitation stations (Fig. 6). In developing the Thiessen polygons, construction lines are drawn between neighboring stations and perpendicular bisectors of these lines are erected as required to obtain closed polygons surrounding the respective precipitation stations. The area (Ap) within each polygon and within the drainage basin is measured and tabulated (Table 4, Col. 4).

(*d*) The average depth of rainfall (Pav) within each station polygon is estimated from the total-storm isohyetal map and tabulated as shown in Part I of Table 4, Col. 6.

(*e*) Rainfall quantities in inches for successive units of time are scaled from mass rainfall curves for the respective precipitation stations. In selecting the "unit" of time to be used, the density of observation stations and character and accuracy of records should be considered. A unit period of 3 hours is suitable for most studies relating to large drainage basins.

(*f*) The effective area (Ae) equal to $(Pav/Psta)(Ap)$ is computed for each precipitation station (Part I, Table 4, Col. 8), and the volume of rainfall within each station polygon, expressed in inch-square miles, is computed by multiplying the 3-hour rainfall quantities obtained in step *e* by Ae (see Part I, Table 4, Cols. 10–24).

(*g*) The initial loss measured in depth in inches is estimated and converted to inch-square miles for each station polygon (Part II, Table 4, Cols. 5, 6). The accumulative rainfall must exceed the initial loss before the infiltration theory is assumed to apply.

(*h*) A trial value of the infiltration index (Fav) in inches per hour is assumed, and the equivalent infiltration loss in inch-square miles for each station polygon is tabulated (Part II, Table 4, Col. 8). The rainfall quantities in excess of the trial value of Fav, after the initial loss has been satisfied, are added and compared with the quantity of surface runoff computed in step *a*. The procedure is repeated until the value of Fav necessary to give rainfall-excess equal to the correct volume of surface runoff is determined. The rainfall and rainfall-excess data for representative zones are plotted as hyetographs above the runoff hydrograph in the manner illustrated in Fig. 8.

The method outlined in the preceding paragraph is not difficult after a reasonable amount of experience in its use has been gained. The basic data necessary are usually required in connection with other phases of hydrologic studies. Inasmuch as the procedure takes into account the areal distribution of rainfall and variations in intensity, it is suitable for application to large drainage basins. The method is susceptible to refinement or simplification, according to the accuracy of basic data and the needs of the particular project. In the example cited, the infiltration index was assumed to be constant over the drainage area, but variations may be estimated by consideration of soil characteristics if the necessary information is available. The probable decrease in infiltration capacities within the respective polygons may also be estimated to correspond with infiltration curves derived from experimental

projects. In some cases, a better representation of rainfall-intensity variations in various portions of a drainage basin may be obtained by interpolating mass rainfall curves for the various areas instead of adhering strictly to the Thiessen-polygon method.

Values of infiltration indices for a large number of drainage basins are presented in Table 5. The indices were computed from data relating to moderate and major floods. The infiltration indices were not derived in exactly the manner outlined in the preceding paragraphs, but the values are considered as an approximate indication of the minimum infiltration indices to be expected during major storms in the respective areas.

D. Regimen of Runoff

17. General. Factors determining hydrographs of runoff have been previously enumerated. The most practical method of estimating the regimen of runoff in natural drainage basins of less than a few thousand square miles in area usually involves an application of the unit hydrograph method, possibly with certain supplementary computations. In estimating the order of runoff from drainage areas greater than a few thousand square miles, a more rational approach consists in estimating runoff from principal tributaries individually by the unit hydrograph method and combining the tributary flows by means of flood-routing computations.

Methods for routing flood flows through natural river channels have been presented in detail by several writers (Art. 65, Refs. 8, 9, 10) and will not be discussed herein. Procedures for the analysis of recorded hydrographs and reapplication of the data obtained in the development of hypothetical hydrographs are suggested in the following paragraphs.

18. Components of Natural Hydrographs. For the purpose of analysis, natural hydrographs may be subdivided into three types of flow, namely:

(*a*) Surface runoff, or the water reaching surface channels by the overland route.

(*b*) Subsurface storm flow, or the portion of infiltrated water that passes through the shallower zones of the soil to reach defined stream channels within a relatively short time after a storm, without having reached the main ground-water table.

(*c*) Ground-water flow, or the water contributed as underground flow from the ground-water table created by infiltration antecedent to the runoff period under study, supplemented by any recharge resulting from penetration of infiltrated water during the period under study.

Until recently it has been common practice to consider runoff as made up of surface runoff and ground-water flow. However, several investigators have observed evidences that an appreciable quantity of water that enters the ground by infiltration during a storm emerges as a direct contribution to stream flow within a relatively short time, and before the water could be expected to have penetrated to the ground-water table and reached the stream

as base flow (Art. 65, Refs. 6, 11). This phenomenon is most evident in the Great Plains region and in certain basins in the midwestern and western United States, but may be observed in varying degrees in other sections of the country.

No definite procedure can be suggested at this time for estimating the volume or rate of runoff from subsurface storm flow where a large proportion of runoff is of that type. A detailed study of basin characteristics and available hydrologic data will usually reveal practical methods of estimating subsurface flow conditions in a particular area. The limited studies of such problems now available indicate that the proper approach in determining both volume and rate of runoff in basins having pronounced subsurface flow characteristics requires a correlation of runoff phenomena with initial field moisture conditions and ground-water storage.

In most of the areas classified as humid in the United States, subsurface storm flow constitutes a small proportion of major flood runoff volumes, although the contributions of subsurface flow to minor flood rises are frequently recognizable and may constitute a significant proportion thereof. In the following discussions of hydrograph analyses related to the development of unit hydrographs or computation of infiltration indices for drainage basins in humid regions, subsurface storm flow will be treated as a part of base flow.

19. Normal Recession Curves. In the analyses of hydrographs it is usually necessary to correct the recorded hydrograph to eliminate runoff from rainfall antecedent or subsequent to the period of rainfall under consideration. In studies of runoff from large basins, flood-routing computations may be required to associate runoff with related rainfall. However, in basins ordinarily involved in unit hydrograph analyses, runoff from distinct periods of intense rainfall can usually be isolated satisfactorily by use of "normal recession curves." A normal recession curve may be computed from segments of hydrographs that represent discharge from natural valley or channel storage, after base flow has been subtracted, as suggested by W. B. Langbein (Art. 65, Ref. 9). Segments of several hydrographs may be necessary to cover a satisfactory range in discharges. Having selected some convenient unit of time, say 6 hr, the discharge at successive time intervals is read from the segments of hydrographs that appear to be unaffected by current rainfall. The discharge values at the beginning of each unit runoff period are plotted against values for the ends of the respective periods. A curve is then drawn as an approximate mean of the highest points, neglecting a few points that plot so far to the left as to appear to represent recession of runoff from a partial-area storm. Values scaled from the mean curve are replotted to the scale of the natural hydrographs to be studied. It should be observed that recession curves derived in the manner described above apply to normal or average storm distributions. It may be expected that runoff from rainfall concentrated in the upper portion of a basin will recede more gradually than indicated by the normal recession curve and conversely for rainfall concentrated in the lower basin.

20. Ground-Water Depletion Curves. The rate of discharge from ground-water storage may be estimated by means of a "ground-water depletion curve" derived from segments of recorded hydrographs that are not appreciably affected by direct runoff or discharge from channel storage. The method suggested in the preceding paragraph for constructing normal recession curves may be used in computing ground-water depletion curves, except that a unit time interval of approximately 2 to 5 days would be used instead of one of a few hours. The distinction between a "normal recession curve," which represents discharge from channel and valley storage, and "ground-water depletion curve," which represents outflow from ground-water storage, should not be overlooked.

21. Subdivision of Hydrographs. A considerable amount of personal judgment is involved in the subdivision of observed hydrographs into the three components of surface runoff, subsurface storm flow, and ground-water flow. The following general procedure is suggested for problems in which the volume of subsurface storm flow is relatively small in proportion to surface runoff from major floods and hence may be satisfactorily considered as a part of base flow:

(*a*) Rainfall data are analyzed by means of isohyetal maps and mass rainfall curves for the entire period of the given flood rise and for several days antecedent thereto. Hyetographs for representative areas are plotted above the hydrograph in proper time relation for use in estimating the timing of surface runoff. (See Fig. 8.)

(*b*) The runoff from rainfall antecedent to the rise under consideration is excluded by extending the recession curve of the antecedent rise to an intersection with the estimated base flow line, as indicated by curve D–C in Fig. 8.

(*c*) The ground-water flow is assumed to decrease in the order indicated by the "normal depletion curve" for a period of approximately 12 to 18 hr after the beginning of the flood rise. (See curve A–B, Fig. 8.) It is probable that during the early periods of a sudden rise a reverse head is imposed on the ground-water table near the stream which must be overcome before an increase in base flow will occur. The first accretion to base flow probably results from recharge by infiltration near the stream channel where the distance to the water table is a minimum. The position of the base flow line must be estimated as well as possible by reference to the last point where it is known that runoff was substantially unaffected by direct runoff from rainfall.

(*d*) A straight line is drawn from the low point of the base flow line to intersect the recession curve of the given rise at a point where it is estimated that discharge resulting from channel storage occasioned by surface runoff had substantially ended. (See curve B–C–E, Fig. 8.) It is intended in this process to include the major portion of subsurface storm flow as a part of base flow. Actually the subsurface storm flow probably reaches a maximum shortly after the rainfall ends, and tends to recede thereafter. However, in view of the many uncertainties involved, the simple procedure suggested for estimating the base flow appears to be satisfactory. Subsurface storm flow may normally be expected to appear in the later parts of a hydrograph, and frequently continues in

recognizable proportions for a period considerably longer than required for surface runoff to pass through the channel-storage phase. A relatively sharp break occurs in the recession curve for some basins, when plotted on logarithmic paper, that appears to indicate the point where the rate of outflow from channel storage becomes relatively small in proportion to subsurface storm flow (Art. 65, Ref. 11). However, such characteristics apparently vary with different basins, and must be ascertained as well as may be feasible from studies of several hydrographs that represent runoff from the basin under a variety of conditions.

E. Unit Hydrographs

22. General. A unit hydrograph, as used herein, is a hydrograph representing 1 in. of runoff from a rainfall of some unit duration and specific areal distribution. The basic premise implies that rainfall-excess of 2 in. within the unit of duration will produce a runoff hydrograph having ordinates twice as great as those of the unit hydrograph. It is also assumed that rates of runoff from consecutive units of rainfall-excess having the same areal distribution will be proportional to the unit hydrograph and that ordinates of the several partial hydrographs obtained by multiplying the unit hydrograph by successive rainfall-excess amounts of unit durations may be added to obtain the total hydrograph of runoff. These basic assumptions are not rigorous, but it has been found by experience that the unit hydrograph method gives results sufficiently accurate for most practical problems, if reasonable judgment is used in its application (Art. 65, Ref. 4).

In the earlier stages of development of the unit-hydrograph method, it was generally assumed that runoff-producing rainfall resulting in a unit hydrograph was uniform over the drainage area involved. However, such a concept greatly restricts the usefulness and applicability of the unit-hydrograph procedure. Unit hydrographs resulting from rainfall-excess quantities uniformly distributed over a drainage basin may be used to compute rates of runoff that would result under average rainfall conditions, whereas one that reflects the regimen of runoff from precipitation of somewhat higher intensity in the lower basin may be useful in estimating critical rates of discharge. Valley storage serves to eliminate the effects of minor variations in rainfall distribution, but major variations in distribution are reflected in the runoff hydrograph. It is practicable to derive unit hydrographs to reflect major variations in rainfall distribution, either by analysis of actual rainfall-runoff records or by use of synthetic methods.

The term "unit-rainfall duration" refers to the duration of runoff-producing rainfall, or rainfall-excess, that results in a unit hydrograph. The unit hydrograph resulting from a 6-hr unit-rainfall duration is referred to as a 6-hr unit hydrograph. The term "lag" as used herein, is the length of time from the midpoint of the unit-rainfall duration to the peak of the unit hydrograph.

The *unit-rainfall duration* selected for a unit hydrograph should not exceed the period during which the design storm rainfall is assumed to be approxi-

mately uniform in intensity in various portions of the drainage area under study. Inasmuch as valley storage tends to eliminate the effects of minor variations in rainfall intensity, somewhat longer unit-rainfall durations are suitable for basins having large valley storage capacities than are otherwise desirable. A 6-hr unit-rainfall duration is suitable and convenient for most studies relating to drainage areas larger than approximately 100 sq mi. Only in approximate studies should unit-rainfall durations longer than 12 hr be adopted, inasmuch as major changes in the areal distribution of rainfall may occur during longer intervals. For drainage areas of less than approximately 100 square miles, values equal to about one-half of the lag appear to be satisfactory.

All references made herein to unit-hydrograph discharge rates, or ordinates, refer to instantaneous discharge values at the instant designated. In order to accurately define the shape of a specific unit hydrograph, any convenient series of discharge ordinates may be tabulated. The interval of time between the discharge ordinates tabulated has no relation whatever to the *unit-rainfall duration* of the particular unit hydrograph. For example, discharge ordinates separated by 6-hr intervals of time are tabulated in Col. 2 of Table 9 for a unit hydrograph resulting from a 12-hr unit-rainfall duration. Discharge ordinates at 12-hr intervals only might have been used to define the same 12-hr unit hydrograph. However, if a 12-hr interval had been used, discharge values of the final hypothetical hydrograph (Col. 10, Table 9) would have been known only at 12-hr intervals, and it would have been necessary to interpolate for intermediate values. Considerable care would have been required in sketching the final hydrograph through known points in order to assure that the correct flood volume was represented. By using 6-hr intervals, a sufficient number of points were obtained to permit an accurate plotting of the final hydrograph, although the ordinates at 12-hr intervals were not changed. It should be observed that a *12-hr unit hydrograph* infers one that results from a unit rainfall-excess of 12 hours' duration, regardless of the interval of time between discharge ordinates used in tabulating the unit-hydrograph values.

A unit hydrograph derived from actual rainfall-runoff records for a particular basin represents an integration of the many influences that affect runoff under the prevailing conditions. Application of such a unit hydrograph to conditions differing from the original is a process of extrapolation, and not an exact mathematical procedure. The validity of the extrapolation should be checked by every means available. A study of unit hydrographs derived for a large number of basins in which a variety of valley storage characteristics, basin configurations, topographical features, and meteorological conditions are represented offers a basis for estimating the relative effects of predominating influences.

The following three general methods are ordinarily available for developing unit hydrographs. In detailed hydrological studies each procedure may be used to advantage.

(*a*) By analysis of rainfall-runoff records for isolated "unit storms."

(*b*) By analysis of rainfall-runoff records for major storms.

(*c*) By computation of synthetic unit hydrographs:

1. From direct analogy with basins of similar characteristics.
2. From indirect analogy with a large number of other basins through the application of empirical relationships.

23. Unit Hydrographs from Isolated Unit Storms. The most direct method of deriving a unit hydrograph involves the analysis of records of runoff resulting from an isolated unit storm that produces reasonably uniform rainfall-excess rates for a period approximately equal to the desired unit-rainfall duration. The following general procedure is suitable for the computations. (See Fig. 9.)

(*a*) Prepare a map (Fig. 9*a*) showing an outline of the basin, the location of stream-gaging stations and precipitation stations in and near the basin.

(*b*) Construct a network of Thiessen polygons (Art. 16–*c*) covering the basin under study.

(*c*) Inspect precipitation records (Weather Bureau Climatological Data) to determine approximately the date of occurrence of periods of intense rainfall over the basin that appear to have been reasonably well isolated from other periods.

(*d*) Refer to stream-flow records for the basin under study to determine approximately the volume of runoff from each of the rainfall periods considered in step *c*. Select for further study the hydrographs that represent at least 1 or 2 in. of runoff.

(*e*) Prepare mass rainfall curves for precipitation stations in and near the basin for each of the periods selected in step *d*. (See Fig. 9*b* and Art. 12.)

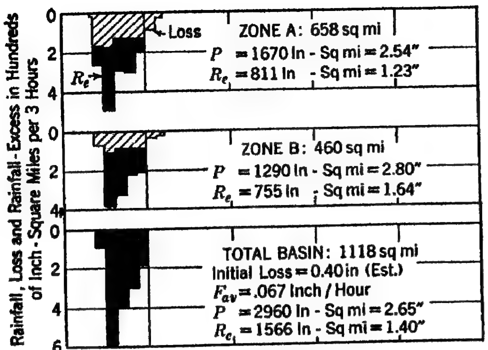
(*f*) Plot discharge hydrographs for each of the periods selected in step *d*. (See Fig. 9*d*.)

(*g*) Study the data obtained in steps *e* and *f* and select for final study those that are most satisfactory for the purpose involved.

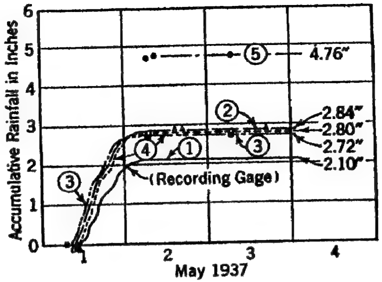
(*h*) Modify the observed hydrographs as required to exclude runoff from extraneous rainfall, and estimate the base flow. Subtract the base flow from the total hydrographs of runoff resulting from the respective unit storms to obtain the hydrographs of surface runoff. (See Fig. 9*d* and Art. 21.)

(*i*) Measure the volume under the hydrographs of surface runoff (by planimetering or computation), compute rainfall-excess quantities, and plot the data in the form of hyetographs. (See Fig. 9*c* and Art. 16.) Although the rainfall-excess data are not directly involved in the computation of unit hydrographs from hydrographs resulting from unit storms, the data are necessary to indicate the areal distribution and intensity characteristics of the runoff-producing rainfall that may have had an important effect on the regimen of runoff.

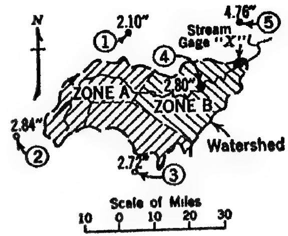
(*j*) Divide the ordinates of the hydrographs of surface runoff resulting from each of the unit storms by the volumes under the respective surface-runoff hydrographs, expressed in inches of runoff from the drainage area, to obtain unit hydrographs. (See Hydrograph D, Fig. 9*c*.)



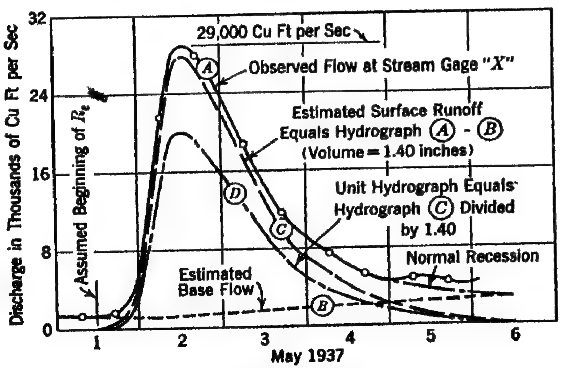
(c) Hyetographs



(b) Mass - Rainfall Curves



(a) Location Map



(d) Hydrographs

Time from Beginning of R_e, in Hours	Instantaneous Discharge, in Cu Ft per Sec
6	300
12	3,100
18	13,300
24	20,000
30	18,600
36	15,900
42	12,600
48	9,600
54	6,800
60	5,300
66	4,000
72	3,000
78	2,300
84	1,800
90	1,300
96	1,000
102	700
108	430
114	230

(e) Tabulation of 12-Hr. Unit Hydrograph (D)

Special Notes

Fig. (a):
Circled Numbers (1), (2), etc. Refer to Precipitation Stations; Station (1) was Equipped with Recording Rain Gage, All Others with Non-Recording Gages.

Fig. (b):
Mass-Curve Numbers Correspond to Precipitation-Station Numbers in Fig. (a). Observed Rainfall Values are Indicated by Legends ∘, ▪, ▴ etc..

Fig. (c):
F_{av} = Average Infiltration Index for Entire Basin.
P = Volume or Average Depth of Precipitation Over Area Designated.
R_e = Volume or Average Depth of Rainfall-Excess Over Area Designated.
= (P) − (Loss by Infiltration, Evaporation etc)

Fig. (d):
Discharge Values Computed from Observed Gage Heights are Indicated by Legend, ∘.
The "Unit-Rainfall Duration" of Unit Hydrograph (D) is Assumed Equal to 12 Hours.

FIG. 9. Computation of unit hydrograph from isolated unit storm.

(*k*) If the durations of rainfall-excess during the various storms differ appreciably from the unit duration adopted for general use, the computed unit hydrographs may be adjusted to the desired unit duration in the manner suggested in Art. 29.

24. Unit Hydrographs from Major Flood Records. The applicability of a unit hydrograph for use in computing the regimen of runoff from an estimated maximum probable rainfall over a basin may be partially verified by reproducing an observed major flood hydrograph by applying the unit hydrograph to rainfall-excess increments of the related storm. The procedure requires a careful analysis of rainfall-runoff data in order to determine major variations in the areal distribution and intensity of rainfall-excess during successive unit periods of the actual storm. Illustrations of data required are presented in Table 4 and Figs. 6, 7, 8, and 13. A unit hydrograph derived from a unit storm, or a trial graph developed by synthetic methods, is first applied to the computed rainfall-excess values to obtain a hypothetical hydrograph for comparison with the observed hydrograph. Modifications in the lag and shape of the unit hydrograph are made as required, following the S-curve procedure discussed in Art. 29, to obtain a reasonably close agreement between the actual and computed hydrographs. In the event major differences exist in the areal distribution of rainfall-excess during successive unit periods of the storm, different unit hydrographs may be applied to the respective rainfall-excess values, using the unit hydrograph corresponding to an areal distribution similar to that of the storm period to which it is to be applied.

The procedure discussed in the preceding paragraph is one method of deriving unit hydrographs. The results obtained generally may be considered more reliable for deriving design floods than those obtained by analysis of minor runoff hydrographs resulting from unit storms, although results of each method should be checked against the other. The fact that a unit hydrograph reproduces a particular major flood hydrograph closely does not necessarily assure that application of the unit hydrograph to design storm rainfall-excess quantities will indicate the critical rates of runoff. The rainfall distribution, intensity, and sequence may be such as to cause a substantially higher rate of runoff during the design storm than is indicated by the unit hydrograph applicable to the actual storm. However, if the ability to account for major flood runoff rates by rational analysis can be demonstrated, greater confidence may be placed in the results obtained in applying the same methods to higher rainfall values.

25. Synthetic Unit Hydrographs—General. In the majority of important hydrologic studies, synthetic unit hydrographs are required either as a substitute for derivations from hydrologic records or as a means of correlating and supplementing observed data. Several methods of computing synthetic unit hydrographs have been presented in technical publications. Most of these methods were developed to serve special purposes and may not constitute the most suitable procedure for certain other uses. For instance, in flood forecasting, the need for speed in calculations may justify approximations that are

not warranted in estimating design flood hydrographs. In estimating critical hydrographs of runoff, conditions favorable to high concentrations of runoff must be assumed, whereas the assumption of average conditions may be more reasonable in other problems. The procedure outlined in the following paragraphs is intended primarily for use in estimating critical rates of runoff from major storms, although the general methods are adaptable to other problems. It is not practicable to eliminate the need for judgment and experience in such studies.

In developing unit hydrographs for use in estimating critical hydrographs of runoff, conservative determinations of (*a*) the peak discharge, (*b*) the degree of concentration of runoff near the peak, and (*c*) the "lag" time are of primary importance. The shape of the rising and recession sides and the length of base of the unit hydrograph are usually of secondary importance if the three components enumerated above are fixed.

26. Snyder's Synthetic Unit-Hydrograph Relations. The empirical relations presented by Franklin F. Snyder (Art. 65. Ref. 7) have proved to be particularly useful in the study of runoff characteristics of drainage areas where stream-flow records are not available, as well as in modifying or supplementing available runoff records to serve specific purposes. The following terms are used in the equations:

t_p = lag time from midpoint of unit-rainfall duration, t_r, to peak of unit hydrograph, in hours.

t_r = unit-rainfall duration equal to $t_p/5.5$, in hours.

t_R = unit-rainfall duration adopted in specific study, in hours.

LAG_{t_R} = lag time from midpoint of unit-rainfall duration, t_R, to peak of unit hydrograph, in hours.

q_{t_R} = peak rate of discharge of unit hydrograph, in cubic feet per second per square mile.

Q_p = peak rate of discharge of unit hydrograph, in cubic feet per second.

A = drainage area, in square miles.

L_{ca} = river mileage from the station to center of gravity of the drainage area.

L = river mileage from the given station to the upstream limits of the drainage area.

C_t and C_p = coefficients depending upon units and drainage basin characteristics.

The following equations are the most frequently used:

$$t_p = C_t(LL_{ca})^{0.3} \quad [8]$$

$$t_r = t_p/5.5 \quad [9]$$

$$LAG_{t_R} = t_p + 0.25(t_R - t_r) \quad [10]$$

$$q_{t_R} = 640\, C_p/LAG_{t_R} \quad [11]$$

$$Q_p = (q_{t_R})(A) \quad [12]$$

The center of gravity of the drainage area above a particular station may be estimated in the following manner:

(*a*) Trace the outline of the drainage basin on a piece of cardboard, and trim to shape.

(*b*) Suspend the cardboard before a plumb bob by means of a pin near the edge of the cardboard, and draw a vertical line. In a similar manner, draw a second line at approximately a 90° angle to the first. The intersection of the two lines is the center of gravity of the area. Transfer the point to the original map for use in determining "L_{ca}."

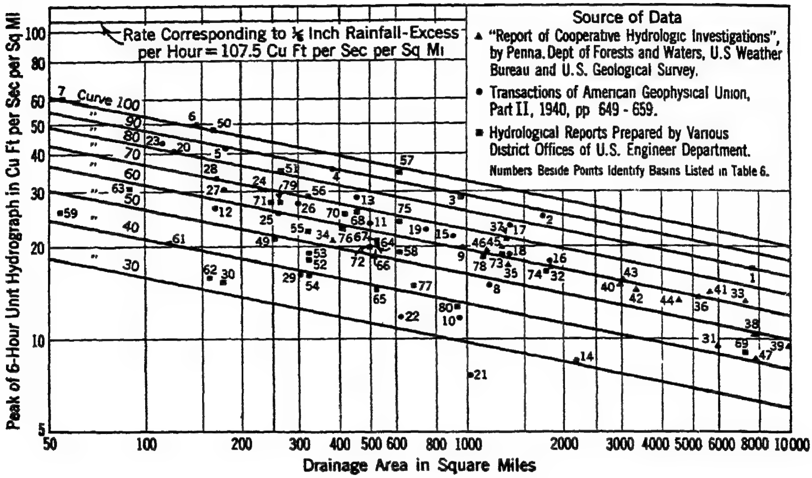


FIG. 10. Six-hour unit-hydrograph peaks versus drainage area.

The distance "L_{ca}" is measured along the principal stream channel to a point approximately opposite the center of the area. The distance "L" is equal to "L_{ca}," plus the remaining distance to the upper basin limit, following approximately the principal stream channel.

The average value of the product 640 C_p is approximately 400, as determined by Snyder for basins in the fairly mountainous Appalachian Highlands, and the corresponding average value of C_t is 2.0. If hydrologic records are available for a representative portion of the drainage area under study, or for nearby basins of similar characteristics, the coefficients C_p and C_t should be computed therefrom. Special field observations to determine the lag may be practicable and desirable in some studies. If the drainage area is formed by two or more important tributaries of different characteristics, unit hydrographs should be computed separately for each and the results combined to obtain a graph for the total area. Reference is made to Snyder's original paper (Art. 65, Ref. 7) for additional details regarding the derivation and use of the equations given above.

27. Unit-Hydrograph Peak Discharges Versus Drainage Area. The peak discharge values of unit hydrographs computed from runoff records for a

large number of basins are plotted against drainage area in Fig. 10. The plotted points are identified in Table 6. As may be expected, a large range in peak-discharge values corresponding to various areas is represented. However, Fig. 10 is useful in comparing synthetic unit hydrograph values with data derived from runoff records for specific basins, or in modifying unit hydrographs derived from runoff records for a particular drainage area to apply to areas that have no stream-flow data available.

28. Concentration of Runoff near Peak. A study of unit hydrographs for a large number of drainage basins has revealed an approximate relationship

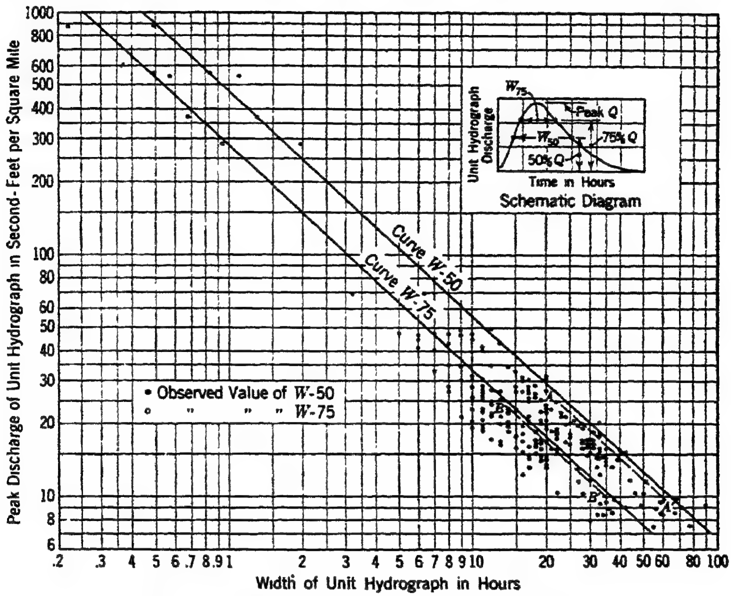


FIG. 11. Unit-hydrograph peaks versus widths.

between the peak discharge rate and the width of unit hydrographs at ordinates exceeding approximately 50 per cent of the maximum. Results of the study are shown in Fig. 11. Curves W-50 and W-75 in Fig. 11 were drawn to envelop the majority of values of unit hydrograph widths measured at discharge ordinates equal to 50 and 75 per cent of the peak ordinate, respectively, as obtained from the study of a large number of unit hydrographs for drainage basins of various configurations and runoff characteristics.

If reliable rainfall-runoff records are not available for deriving unit hydrographs for a particular drainage area, curves W-50 and W-75 of Fig. 11 may be used in determining conservative widths for synthetic unit hydrographs having various peak values. In the event unit hydrograph data are available for representative portions of a basin, and it is desired to modify such unit hydrographs to represent more conservative runoff rates, the known width values may be plotted on Fig. 11 and curves applicable to the specific basin

TABLE 6

IDENTIFICATION OF DRAINAGE AREAS CONSIDERED IN PREPARATION OF FIG. 10

Point No.	*Stream*	*Location*
1	Washita River, Okla.	Near mouth
2	Youghiogheny River	Sutersville, Pa.
3	Black River	Leeper, Ark.
4	Casselman River	Markleton, Pa.
5	South Fork Ten Mile Creek	Jefferson, Pa.
6	Turtle Creek	East Pittsburgh, Pa.
7	Canacadea Creek	Almond dam site, N. Y.
8	Tygart River	Tygart dam site, W. Va.
9	Cheat River	Rowlesburg, W. Va.
10	Allegheny River	Kinzua, Pa.
11	Little Beaver Creek	East Liverpool, Pa
12	Sugar Creek	Sugar Creek, Pa.
13	Redbank Creek	Redbank Creek dam site, Pa.
14	Allegheny River	Above Kinzua, Pa.
15	Clarion River	Clarion, Pa.
16	Kiskiminitas River	Vandergrift, Pa.
17	Cheat River	Beaver Hole, W. Va.
18	Conemaugh River	Bow, Pa.
19	West Fork River	Enterprise, W. Va.
20	Laurel Hill Creek	Ursina, Pa.
21	French Creek	Utica, Pa.
22	French Creek	Saegerstown, Pa.
23	Buffalo Creek	Barrackville, W. Va.
24	Dunkard Creek	Bobtown, Pa.
25	Chartiers Creek	Carnegie, Pa.
26	Oil Creek	Rouseville, Pa.
27	Raccoon Creek	Moffatts Mills, Pa.
28	Yellow Creek	Hammondsville, Pa.
29	Brokenstraw Creek	Youngsville, Pa.
30	Millers River	Birch Hill, Mass.
31	Allegheny River	Franklin, Pa.
32	Allegheny River	Vandergrift, Pa.
33	Monongahela River	Dam No. 2, Pa.
34	West Fork River	Clarksburg, W. Va.
35	Tygart River	Fetterman, W. Va.
36	Monongahela River	Charleroi, Pa.
37	Youghiogheny River	Connellsville, Pa.
38	Susquehanna River	Towanda, Pa.
39	Susquehanna River	Wilkes-Barre, Pa.
40	West Branch Susquehanna River	Renova, Pa.
41	West Branch Susquehanna River	Williamsport, Pa.
42	Juniata River	Newport, Pa.
43	Delaware River	Port Jervis, N. Y.
44	Delaware River	Belvidere, N. J.
45	Lehigh River	Bethlehem, Pa.
46	Schuylkill River	Pottstown, Pa.
47	Susquehanna River	Towanda, Pa.
48	Canisteo River	Arkport Dam, N. Y.
49	Otselic River	Whitney Point, N. Y.
50	Westfield River	Knightville dam site, Mass.
51	Loyalhanna Creek	New Alexandria, Pa.
52–56	Mahoning Creek	Dayton, Pa.
57–58	Coldwater River	Coldwater, Miss.
59	Saddle River	Lodi, N. J.
60	Whippany River	Morristown, N. J.
61	Ramapo River	Mahwah, N. J.
62	Ramapo River	Pompton Lakes, N. J.
63	Wanaque River	Wanaque, N. J.
64–68	Redbank Creek	Pennsylvania
69	Washita River	Durwood, Okla.
70	Strawberry River	Poughkeepsie, Ark.
71	Petit Jean River	Booneville, Ark.
72	Petit Jean River	Blue Mountain dam site, Ark
73	North Branch White River	Tecumseh, Mo.
74	North Branch White River	Norfork dam site, Ark.
75	Eleven Point River	Bardley, Mo.
76	Fourche la Fave River	Gravelly, Ark.
77	Fourche la Fave River	Nimrod dam site, Ark.
78	Little Red River	Greer's Ferry, Ark.
79	Row-Willamette River	Dorena (Star), Oreg.
80	Illinois River	Tahlaquah, Okla.

may be drawn through these points parallel to curves W-50 and W-75, as illustrated by curves A–A′ and B–B′, respectively.

29. S-Curve Hydrographs. According to the unit hydrograph concept, if the unit rate of rainfall-excess over a drainage area should continue indefinitely with the same areal distribution and intensity characteristics, successive units of rainfall-excess would contribute runoff at rates corresponding to the basic unit hydrograph. An accumulation of runoff ordinates corresponding to a particular time would give the total rate of runoff produced by the uniform

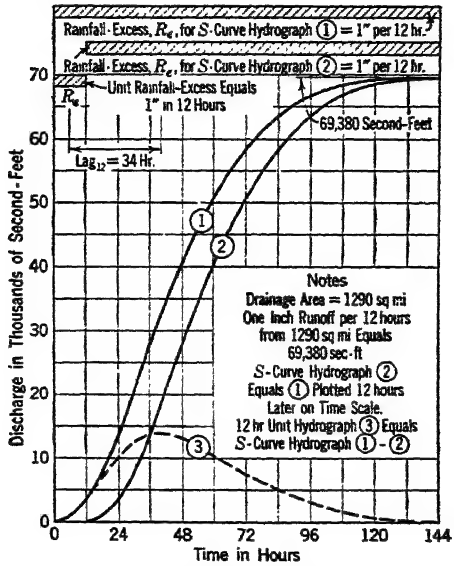


FIG. 12. Relation of unit hydrograph to S-curve hydrograph.

continuous rate of rainfall-excess antecedent thereto. At a time equal to the base of the unit hydrograph the rate of runoff would become equal to the rate of rainfall-excess and would remain constant thereafter. The hydrograph generated in this fashion will be referred to herein as an S-curve hydrograph. (See Fig. 12.) The S-curve hydrograph as defined above should not be confused with mass curves of runoff that represent accumulative volumes resulting from varying rates of rainfall-excess.

An S-curve hydrograph may be computed by tabulating a series of identical unit hydrographs, arranged with origins spaced progressively one unit-rainfall duration apart on the time scale and accumulating the ordinates for specific times. A more convenient procedure is illustrated in Table 7 and Fig. 12. With the unit hydrograph values known, the S-curve hydrograph would be computed by steps. During the first 12-hr unit-rainfall duration, the unit hydrograph (Col. 2, Table 7) and S-curve values (Col. 4) are identical. The S-curve values shown in Col. 4 of Table 7 for the first 12-hr period are trans-

TABLE 7

RELATION OF UNIT HYDROGRAPHS TO S-CURVE HYDROGRAPHS *

Time in Hours	Computation of S-Curve Hydrograph from Known 12-Hour Unit Hydrograph			Computation of 6-Hour Unit Hydrograph from 12-Hour S-Curve Hydrograph		
	12-Hour Unit Hydrograph ③, Fig. 12, in C.F.S.	12-Hour S-Curve Hydrograph ②, Fig. 12, in C.F.S.	12-Hour S-Curve Hydrograph ①, Fig. 12, in C.F.S.	12-Hour S-Curve Hydrograph ① Shifted 6 Hours	Runoff from 0.5 Inch R_c in 6 Hours (Cols. 4–5)	6-Hour Unit Hydrograph (2 times Col. 6) in C.F.S.
1	2	3	4	5	6	7
6	900		900		900	1,800
12	3,400		3,400	900	2,500	5,000
18	6,900	900	7,800	3,400	4,400	8,800
24	10,100	3,400	13,500	7,800	5,700	11,400
30	12,300	7,800	20,100	13,500	6,600	13,200
36	13,600	13,500	27,100	20,100	7,000	14,000
42	13,900	20,100	34,000	27,100	6,900	13,800
48	13,200	27,100	40,300	34,000	6,300	12,600
54	11,800	34,000	45,800	40,300	5,500	11,000
60	10,300	40,300	50,600	45,800	4,800	9,600
66	8,950	45,800	54,750	50,600	4,150	8,300
72	7,650	50,600	58,250	54,750	3,500	7,000
78	6,400	54,750	61,150	58,250	2,900	5,800
84	5,250	58,250	63,500	61,150	2,350	4,700
90	4,200	61,150	65,350	63,500	1,850	3,700
96	3,200	63,500	66,700	65,350	1,350	2,700
102	2,280	65,350	67,630	66,700	930	1,860
108	1,580	66,700	68,280	67,630	650	1,300
114	1,100	67,630	68,730	68,280	450	900
120	750	68,280	69,030	68,730	300	600
126	500	68,730	69,230	69,030	200	400
132	300	69,030	69,330	69,230	100	200
138	150	69,230	69,380	69,330	50	100
144	50	69,330	69,380	69,380	0	0

* All discharges are instantaneous values at the end of the hour designated in Col. 1. Drainage area = 1290 sq mi. (See also Fig. 12.)

ferred to Col. 3 and added to unit hydrograph discharges for the second 12-hr interval to obtain corresponding S-curve ordinates. The process is continued until the S-curve discharge rate is equal to the basic rate of rainfall-excess. The operations involved are more apparent if the reverse procedure is considered, assuming that the S-curve hydrograph values tabulated in Col. 4 were originally known. Discharge values shown in Col. 4 represent the rates of runoff that would result from a uniform continuous rate of rainfall-excess of 1 in. in 12 hr, beginning at time zero. If the discharge values shown in Col. 4 are entered in Col. 3, with the origin time 12 hr later, the difference between values in Cols. 4 and 3 will represent the rate of runoff from 1 in. of rainfall-excess in 12 hr (Col. 2). (See Fig. 12.)

In accordance with the unit hydrograph principle, the ordinates of an S-curve hydrograph representing runoff from a uniform continuous rainfall-excess rate of 1 in. per 12 hr may be multiplied by 2 in order to obtain values applicable to a rainfall-excess rate of 1 in. per 6 hr. Accordingly, S-curve hydrographs developed from runoff data for unit storms of various durations may be adjusted to apply to any unit rainfall duration desired, within practical limits. The computation of a 6-hr unit hydrograph from a 12-hr S-curve hydrograph is illustrated in Cols. 5–7 of Table 7.

In addition to the applications referred to above, the S-curve procedure is useful in modifying unit hydrographs to represent more conservative peak values, or to reflect moderate changes in rainfall distribution, as discussed hereinafter.

30. Summary of Synthetic Unit-Hydrograph Computations. In developing unit hydrographs for use in computing hypothetical hydrographs of runoff from major storms, without the benefit of reliable and adequate rainfall-runoff data, the following general procedure is recommended:

(*a*) Analyze such hydrologic data as are available for portions of the drainage area having stream-flow records to determine approximately the peak discharge, lag, and general shape of unit hydrographs. In many instances, fragmentary hydrologic data that are not adequate for unit hydrograph derivation in the usual manner may be very useful in connection with synthetic analyses.

(*b*) If adequate hydrologic records are available for a representative portion of the drainage basin, evaluate coefficients C_p and C_t in Eqs. 8 and 11 of Art. 26, and use these values in estimating the peak discharge of a synthetic unit hydrograph for the given drainage area. Lacking hydrologic records for evaluating C_p and C_t, tentatively adopt the average values given by Snyder for the coefficients.

(*c*) By a general comparison of runoff characteristics involved, estimate whether the unit-hydrograph peak-discharge values computed for the particular area are consistent with values shown in Fig. 10 for comparable basins.

(*d*) Having decided upon a conservative value for the peak-discharge rate of the unit hydrograph, complete the computation of the synthetic unit hydrograph in the manner outlined in the subsequent paragraph on "Unit-Hydrograph Adjustments."

31. Unit-Hydrograph Adjustments. Following is an illustration of the procedure used in adjusting the 6-hr unit hydrograph No. 1 of Fig. 13, derived from hydrologic records for the 1290 sq-mi drainage area of the Saluda River above Chappells, S. C., to apply to the 970 sq-mi drainage area represented by subarea No. 1 of Fig. 15. (See Table 8.)

(*a*) The widths of unit hydrograph No. 1 of Fig. 13 were scaled at ordinates equal to 50 and 75 per cent of the peak discharge, respectively, the values were plotted in Fig. 11, and lines A–A′ and B–B′ were drawn through the points parallel to curves W-50 and W-75 shown thereon.

(*b*) Discharge ordinates of unit hydrograph No. 1, Fig. 13, were reduced in direct proportion to the drainage areas involved (970/1290 = 0.75), and the reduced values were plotted as hydrograph No. 1 in Fig. 14 to serve as a guide in shaping the modified unit hydrograph.

(*c*) Coefficients C_p and C_t, corresponding to unit hydrograph No. 1 for the 1290 sq-mi drainage area above Chappells, S. C., were computed as follows:

$$L_{ca} = 47 \text{ mi (measured from map)}$$

$$L = 92 \text{ mi (measured from map)}$$

$$(LL_{ca})^{0.3} = 12.3$$

$$LAG_{t_R} = 34 \text{ hr (see Fig. 13)}$$

$$t_R = 6 \text{ hr (selected value)}$$

$$= t_r, \text{ to nearest hour}$$

$$Q_{t_R} = 14{,}100 \text{ cu ft per sec (see Fig. 13)}$$

$$q_{t_R} = 14{,}100/1290$$

$$= 10.9 \text{ cu ft per sec per sq mi}$$

$$C_t = LAG_{t_R}/(LL_{ca})^{0.3} \text{ (approximate)}$$

$$= 34/12.3$$

$$= 2.8 \text{ (see Eq. 8)}$$

$$640\ C_p = (LAG_{t_R})(q_{t_R})$$

$$= 370 \text{ (see Eq. 11)}$$

(*d*) Assuming that coefficients C_t and C_p computed for the 1290 sq-mi drainage area are applicable to the 970 sq-mi area under study, the peak

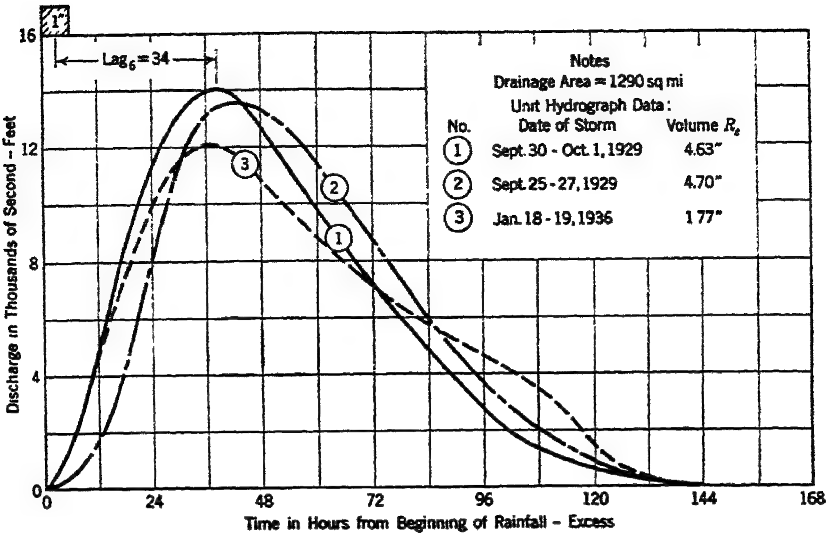


FIG. 13. Six-hour unit hydrographs, Saluda River at Chappells, S. C.

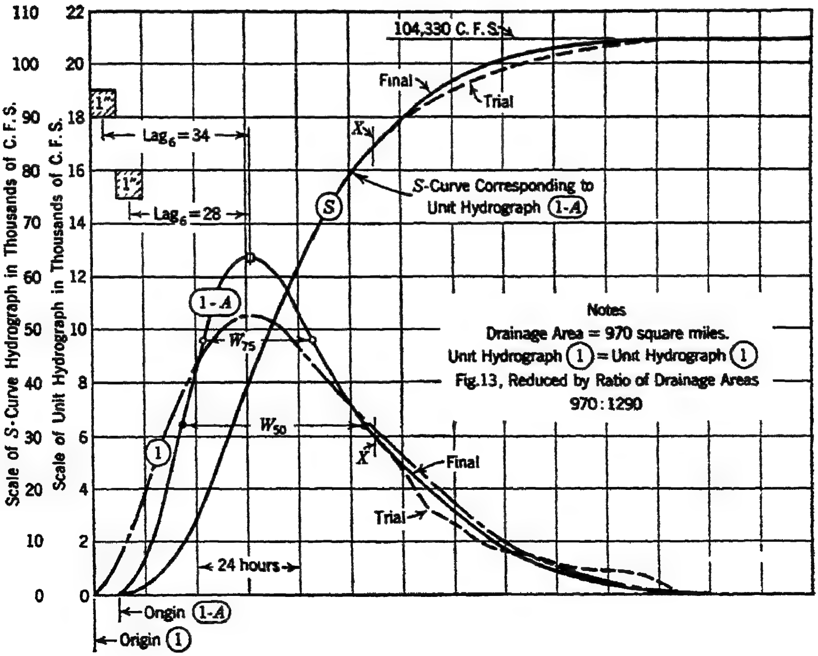


FIG. 14. Adjustment of unit hydrograph for change in drainage area.

TABLE 8

COMPUTATION OF UNIT HYDROGRAPH

(ILLUSTRATION) Reservoir; SALUDA RIVER Basin, S.C.

Location SUB-AREA NO. 1, FIG. 15 ; D.A. 970 sq. mi.; selected t_R 6 Hrs.

Snyder's Eqs: L 66 ; L_{ca} 32 ; $(LL_{ca})^{0.3}$ 9.9 ; C_t 2.8 ; t_p 27.7

$t_r = (t_p/5.5) =$ 5.1 ; $0.25\,(t_R - t_r) =$ 0.2 ; $LAG_{tR} =$ 28 ; 640 C_p = 370

$q_{tR} =$ 13.2 C.F.S./sq. mi.; $Q_p =$ 12,800 C.F.S.; Peak-Area Rating in % 41

S-Curve: $Q_{max.} = (D.A.)\,(26.89)\,(24/t_R) = (D.A./t_R)\,(645.36) =$ 104,330 C.F.S.

Line No.	Graph No. 1-A (a) Trial Values				Graph No. 1-A (b) Final Values			
	Time in Hours	q_{tR} 13.2; LAG 28; Rating 41 W_{75} 27; W_{50} 44; Base 126			Time in Hours	q_{tR} 13.2; LAG 28; Rating 41 W_{75} 27; W_{50} 44; Base 126		
		Unit Hydrograph	S-Curve Subtraction	S-Curve Hydrograph		Unit Hydrograph	S-Curve Subtraction	S-Curve Hydrograph
1	2	3	4	5	6	7	8	9
1	6	1,300		1,300	6	1,300		1,300
2	12	4,400	1,300	5,700	12	4,400	1,300	5,700
3	18	8,200	5,700	13,900	18	8,200	5,700	13,900
4	24	11,700	13,900	25,600	24	11,700	13,900	25,600
5	30	12,800	25,600	38,400	30	12,800	25,600	38,400
6	36	12,400	38,400	50,800	36	12,400	38,400	50,800
7	42	11,000	50,800	61,800	42	11,000	50,800	61,800
8	48	9,000	61,800	70,800	48	9,000	61,800	70,800
9	54	7,400	70,800	78,200	54	7,400	70,800	78,200
10	60	6,000	78,200	84,200	60	6,000	78,200	84,200
11	66	4,800	84,200	89,000	66	5,000	84,200	89,200
12	72	3,300	89,000	92,300	72	4,100	89,200	93,300
13	78	2,700	92,300	95,000	78	3,200	93,300	96,500
14	84	2,000	95,000	97,000	84	2,500	96,500	99,000
15	90	1,700	97,000	98,700	90	1,800	99,000	100,800
16	96	1,300	98,700	100,000	96	1,300	100,800	102,100
17	102	1,100	100,000	101,100	102	900	102,100	103,000
18	108	1,000	101,100	102,100	108	550	103,000	103,550
19	114	900	102,100	103,000	114	400	103,550	103,950
20	120	900	103,000	103,900	120	250	103,950	104,200
21	126	430	103,900	104,330	126	130	104,200	104,330

Note: All discharges are instantaneous values corresponding to end of hour designated in Col. 2 Reference is also made to the curves shown in Fig. 14.

discharge and lag corresponding to the smaller area were computed as follows:

$$L_{ca} = 45 \text{ mi (measured from map)}$$

$$L = 85 \text{ mi (measured from map)}$$

$$(LL_{ca})^{0.3} = 9.9$$

$$LAG_{t_R} = C_t(LL_{ca})^{0.3}$$

$$= 2.8\ (9.9)$$

$$= 28 \text{ hr}$$

$$q_{t_R} = 640\ C_p/LAG_{t_R}$$

$$= 370/28$$

$$= 13.2 \text{ cu ft per sec per sq mi}$$

$$Q_{t_R} = 12{,}800 \text{ cu ft per sec (see unit hydrograph 1-A of Fig. 14)}$$

(*e*) Values of W_{50} and W_{75} corresponding to the synthetic unit-hydrograph peak value computed for the 970 sq-mi area above the dam site were read from curves A–A′ and B–B′ of Fig. 11, and the respective values were indicated by points plotted on each side of the unit-hydrograph peak, approximately as shown in Fig. 14.

(*f*) A tentative synthetic unit hydrograph was sketched through the estimated peak discharge, and the plotted values of W_{50} and W_{75}, terminating temporarily at approximately point X indicated in Fig. 14.

(*g*) A provisional S-curve hydrograph corresponding to tentative unit hydrograph No. 1-A was computed to point X and projected forward as a smooth curve until the maximum ordinate was reached. The computation of the trial unit hydrograph was then completed, as indicated in Table 8, Part (a). Minor adjustments were made in the provisional unit hydrographs and S-curve hydrograph until the most logical forms of both were obtained, as indicated by the final curves of Fig. 14. The final computations are shown in Table 8, Part (b). In making the adjustments, it is convenient to work from the right end of the unit hydrograph where the correct S-curve value is known, and work backward toward point X by assuming values of unit-hydrograph ordinates that appear reasonable and computing the corresponding S-curve values. The adjustment necessary to make the two portions of the S-curve meet near point X can easily be made.

32. Comparison of Unit Hydrographs Derived from Major and Minor Flood Hydrographs. The definition of the unit hydrograph implies that ordinates of any hydrograph resulting from a quantity of runoff-producing rainfall of unit duration would be equal to corresponding ordinates of a unit hydrograph for the same areal distribution of rainfall, multiplied by the ratio of rain-

fall-excess values. However, the relation inferred by the definition is only approximately correct and may be appreciably in error if the conditions affecting runoff differ greatly during floods of various magnitudes in the basin under study.

In an effort to determine the probable degree of accuracy inherent in the use of unit hydrographs derived from records of minor floods in estimating the critical rates of runoff from maximum probable storms, hydrologic data for minor and major floods of record in a large number of basins have been analyzed. Minor floods were selected that resulted from rainfall of relatively uniform areal distribution. The volumes of rainfall and rainfall-excess during successive 6-hr periods of storms causing major floods in each basin were computed, and unit hydrographs were developed that would reproduce the observed hydrograph when applied to the known rainfall-excess values. Most of the major floods investigated were the result of one or more periods of intense rainfall of approximately 12-hr duration, supplemented by periods of lighter precipitation. The same general procedures were followed in the analysis of records and in the computation of unit hydrographs for both minor and major floods, insofar as the character of basic data permitted. The topography of the basins considered varied from rolling slopes to relatively steep hills several hundred feet in height above the principal stream channels.

With but a few exceptions, it was found that unit hydrographs required to reproduce the major flood hydrographs had peak-discharge ordinates consistently higher than those computed from records of minor floods in which the areal distribution of rainfall was approximately uniform. In most of the basins considered, the peak ordinates of unit hydrographs derived from major flood hydrographs, representing runoff volumes greater than approximately 5 in. in depth from the drainage area, were 25 to 50 per cent higher than values computed from records of minor floods, in which the runoff was from 1 to 2 in. The variations were greater in a few instances. The differences were not proportional to the volumes of flood runoff but apparently were the result of a number of factors, some of which had greater influences during certain floods than during others.

The following probably represent the principal reasons for the observed differences between unit hydrographs derived from minor and major flood hydrographs referred to above:

(*a*) *Differences in areal distribution of rainfall.* The minor flood rises analyzed resulted from rainfall of approximately uniform areal distribution. Precipitation during the major floods usually covered the entire drainage area, but in general the rainfall intensity and accumulated amounts varied over the area. If the volume of rainfall-excess during the major storm was proportionately heavier in the lower part of the basin, or near the principal stream channels, the concentration of runoff would be higher than represented by the unit hydrographs derived from the minor flood rises.

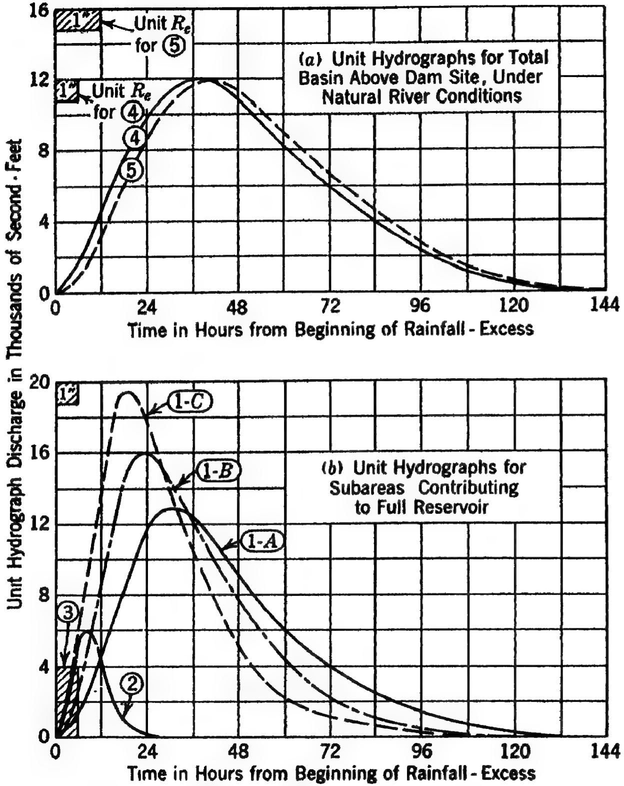
(*b*) *Differences in hydraulic relations.* During minor flood rises, the hydraulic gradients in natural streams are usually relatively low, because of

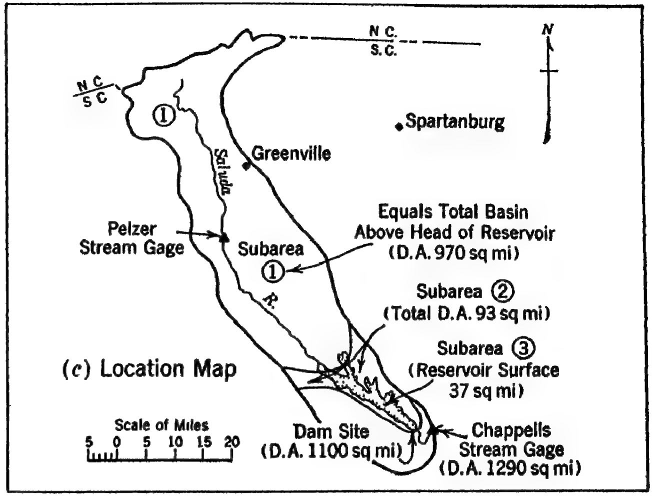
the series of pools that exist in the channels. As the stage increases during major floods, the pools tend to drown out and, in basins less than several hundred square miles in area, the channel conveyance is usually increased substantially. It is therefore to be expected that unit hydrographs derived from major flood hydrographs for small streams would have higher peak-discharge ordinates than those derived from minor floods. In large basins in which great quantities of overbank storage occur at flood stages, the channel conveyance may decrease with rises in stage but the higher efficiencies of flow in tributary streams tend to counteract such decreases.

33. Selection of Unit Hydrographs for Design-Flood Computations. In the majority of hydrologic studies pertaining to the determination of maximum probable flood hydrographs, reliable data for the determination of unit hydrographs are limited to records of relatively minor flood occurrences. For reasons discussed in Art. 32, it is usually justifiable to assume that a unit hydrograph applicable to the most intense periods of rainfall during a design storm would have a higher peak-discharge ordinate, and would represent a higher concentration of runoff, than might be indicated by unit hydrographs derived from minor floods. If adequate and reliable hydrologic records are available for floods that resulted from rainfall intensities and areal distributions reasonably comparable to those to be expected during the design storm, uncertainties regarding the proper unit-hydrograph values for use in estimating the design flood runoff are substantially reduced. However, it is frequently necessary to modify unit hydrographs derived from available hydrologic records to represent higher rates of runoff, in order that their use in computing the design flood discharges may assure conservative results.

The general procedure outlined in Art. 31 is suitable for modifying a given unit hydrograph to represent a higher peak discharge. The method was followed in arbitrarily modifying unit hydrograph No. 1-A, Fig. 15*b*, to obtain Nos. 1-B and 1-C, which have peak ordinates 25 and 50 per cent higher, respectively, than No. 1-A. The widths of each of the three unit hydrographs at ordinates equal to 50 and 75 per cent of the maxima, were obtained from curves A–A′ and B–B′ of Fig. 11.

The unit hydrographs selected for estimating rates of runoff from successive unit periods of the design storm should be applicable to the areal distributions of rainfall that are assumed to occur during the respective periods. As a general rule, it is ordinarily satisfactory to consider that rainfall during periods of relatively moderate intensities is approximately uniform in areal distribution and that the regimen of runoff therefrom may be estimated by application of a unit hydrograph derived from minor flood records. However, during the most intense 12-hr period of rainfall, it is reasonable to assume that the depth of rainfall in the lower basin, or near the principal stream channels, may be greater than the average over the entire drainage area and that the concentration of runoff may be higher, both by reason of a critical distribution of rainfall and by increased hydraulic efficiencies characteristic of higher stages in tributary streams. On the basis of results obtained by comparing unit hydro-





(d) Data Relating to Unit Hydrographs

No.	Unit-Duration in Hours	Area Represented: Identification	Area Represented: Sq Mi	Remarks
1-A	6	Subarea No. 1	970	Equals Unit Hydrograph 1 Fig. 14
1-B	6	" " 1	970	* Alternate to No. 1-A: Peak = 125% Peak of 1-A
1-C	6	" " 1	970	* Alternate to No. 1-A: Peak = 150% Peak of 1-A
2	6	" " 2	93	Estimated for Sum of Small Areas
3	6	" " 3	37	Rate of Runoff = Rainfall Intensity
4	6	Total Above Dam Site	1100	For Natural River Conditions
5	12	" " " "	1100	" " " "

*Obtained by Arbitrarily Modifying Unit Hydrograph No. 1-A

FIG. 15. Unit hydrographs used in developing spillway design flood criteria.

graphs derived from minor and major floods, as discussed in Art. 32, it is usually reasonable to assume that a unit hydrograph applied to the maximum 12-hr rainfall-excess value of a design storm should have a peak-discharge ordinate 25 to 50 per cent higher than a unit hydrograph representative of runoff from rainfall of moderate intensity and uniform areal distribution. The difference may be greater in certain unit hydrographs.

Decisions regarding the modifications that should be made in a unit hydrograph derived from hydrologic records, in order to assure a conservative estimate of design flood discharges, are primarily dependent upon judgment. The character and scope of basic data, the purpose of the estimate, and the importance of conservative results are matters to be considered. When developing a spillway design flood for a reservoir project, it is usually desirable to determine the amount of increase in the maximum reservoir level that would result from various differences in the concentration of runoff from the design storm before final decisions regarding selection of unit hydrographs are attempted.

34. Reservoir Inflow Unit Hydrograph. The formation of a long reservoir in a natural drainage basin may materially alter the regimen of flood runoff by synchronizing high rates of runoff originating above the head of the reservoir with maximum rates from areas contributing laterally to the reservoir. Under natural river conditions, runoff from the upper portion of a basin is retarded by valley storage and normal frictional resistance as it passes through the reservoir reach, the resultant velocity corresponding to that indicated by Manning's formula for flow in open channels. However, after a deep reservoir has been formed by construction of a dam, inflow near the upper end of the reservoir moves through the pool largely by a process of translation, with long-wave velocities subject to momentum control, equal approximately to $\sqrt{gd}$, in which d is the depth of flow in feet, and g is the acceleration due to gravity (32.2). Estimates of the time required for flood waves to traverse natural river channels within the limits of reservoirs may range from a few hours to approximately 1½ days, whereas the time required for inflow into the upper end of the full reservoir to become effective at the point of reservoir outflow would range from practically zero to a few hours for comparable storms. Changes in the synchronization of runoff from various portions of a drainage basin may be such as to produce rates of inflow into a full reservoir that are substantially higher than would occur at the dam site under natural river conditions, although in some reservoirs the differences may be negligible.

The critical rate of inflow into a full reservoir during the spillway design storm may be conveniently estimated by the following method, which conforms approximately with a procedure developed in the Little Rock District Office of the United States Engineer Department, U. S. Army:

(*a*) The drainage area contributing to the reservoir is divided into subareas in the manner illustrated in Fig. 15*c*.

(*b*) Unit hydrographs are derived for the respective subareas, using such hydrologic records as are available, supplemented by synthetic unit hydro-

graph computations. The rate of runoff from the reservoir surface is taken equal to the rate of rainfall.

(*c*) The time required for flood waters entering the upper end of the reservoir to become effective in raising the reservoir level at the spillway site is estimated by assuming the velocity of translation as equal to $\sqrt{gd}$. If the maximum translation time is of significant length, the time required for runoff from each stream that contributes directly to the reservoir to become effective in raising the reservoir level at the spillway site is estimated, assuming that the time of travel is proportional to distance.

(*d*) Unit hydrographs for the subareas are plotted on the same sheet in the time relation determined in the preceding step. (See Fig. 15*b*.) For convenience in use, unit hydrographs for the several minor subareas immediately adjacent to the reservoir are combined into a single unit hydrograph applicable to the sum of the areas involved. In order that the effects of different areal distributions of rainfall during the spillway design storm may be evaluated, unit hydrographs for the various subareas should be retained in separate form or combined in such manner as to permit the computation of runoff separately for (1) principal tributaries, (2) minor subareas immediately adjacent to the reservoir, and (3) the reservoir surface.

(*e*) Hydrographs of runoff from the various subareas, or groups of subareas, corresponding to the spillway design-storm rainfall-excess quantities, are computed and combined in proper time relation to obtain a composite hydrograph of reservoir inflow. (See Fig. 17.) Hypothetical hydrographs corresponding to several possible distributions of rainfall may be computed in a similar manner in order to determine the critical values.

F. Spillway Design Storms

35. General. The term "spillway design storm" as used herein refers to the rainfall criteria adopted as a basis for the computation of a hydrograph to represent the most critical combination of volume and rate of runoff considered reasonably probable in a particular basin. The spillway design storm may consist of a single period of intense rainfall or any series of rainfall events that may be expected to occur over the area. In other words, the spillway design storm may actually be a series of separate rainstorms, defined according to popular usage of the term, if such a series is reasonably probable and is capable of producing a more critical hydrograph of runoff than a single event.

In determining the critical design-storm rainfall estimate for a particular drainage area, it is necessary to consider the size, configuration, and runoff characteristics of the basin, as well as meteorological characteristics of major storms in the region. In some drainage basins, very intense storms of relatively short duration produce critical discharges, whereas in others storms of less intensity and longer duration result in the most severe floods. As a general rule, meteorological conditions that result in the most intense rainfall

rates over small areas differ from those that cause maximum precipitation over large areas. (For a concise treatise on meteorology, see Art. 65, Ref. 12.)

A rational determination of critical design-storm criteria for a particular drainage basin requires a comprehensive study of major storms of record in the region and an evaluation of the effects of local conditions in the project area upon rainfall and runoff rates. The comprehensive studies may follow approximately the outline given below:

(*a*) Analyze precipitation data and synoptic situations of major storms of record in a region surrounding the basin under study, in order to determine characteristic combinations of meteorological conditions that result in various rainfall patterns and duration-depth-area relations.

(*b*) On the basis of an analysis of air-mass properties and synoptic situations prevailing during the record storms, estimate the amount of increase in rainfall quantities that would have resulted if conditions during the actual storm had been as critical as those considered probable of occurrence in the region.

(*c*) Estimate the modifications in meteorological conditions that would have been required for each of the record storms to have occurred over the drainage basin under study, considering topographic features and locations of the respective areas involved.

(*d*) Taking into account the increase in rainfall quantities that might have resulted from more severe meteorological conditions during the record storms, and the adjustments necessary to transpose the respective storms to the basin under study, select the estimate that would represent critical rainfall duration-depth-area relations for the particular drainage area during various seasons of the year.

(*e*) Estimate the maximum quantity and rate of contribution to flood runoff that might result from melting snow in conjunction with the critical storm rainfall for the snow season.

(*f*) Taking into account the minimum infiltration capacities likely to prevail during the various seasons, and the maximum contribution from melting snow during the snow season, select the design-storm estimate that would result in the critical runoff hydrograph for the project involved. In some projects it may be necessary to compute hydrographs for two or more rainfall estimates to determine the critical combination of volume and rate of runoff.

In view of the broad scope of the studies referred to above it is not practicable to present in this chapter a detailed treatment of the methods involved. For information regarding studies of storm rainfall and runoff from melting snow, reference is made to technical publications and to reports by various State and Federal agencies. (See Art. 65, Ref. 13–24.) It is apparent that the detailed meteorological studies outlined in the preceding paragraph require a comprehensive compilation of data, considerable time for the studies, and the services of experienced personnel. Although the special comprehensive studies are fully justified and highly desirable for important projects, it is frequently necessary to prepare design-storm rainfall estimates on the basis of data imme-

diately available. Certain of the problems involved in the determination of design-storm rainfall estimates, and the methods most commonly followed in their preparation when the results of comprehensive meteorological studies are not available, are discussed in the following paragraphs.

36. Approximate Design-Storm Estimates for Seasons without Snow. Estimates of rainfall quantities to be expected during successive unit periods of a design storm for a drainage basin less than a few thousand square miles in area are usually expressed as average depths over the drainage area, and the critical sequence of intense-rainfall increments is determined arbitrarily by trial applications of unit hydrographs. Normal variations in the areal distribution of design-storm rainfall over small basins may be satisfactorily allowed for in selecting infiltration indices and in developing unit hydrographs to reflect critical conditions. However, in determining the maximum probable flood for large drainage basins, definite estimates or assumptions regarding the areal distribution of rainfall during successive periods of the design storm are necessary. The principal reasons are as follows:

(*a*) The infiltration opportunity during a storm period is proportional to the area covered by rainfall of an intensity greater than the infiltration capacity of the soil. If precipitation during a particular period of the design storm should cover only a portion of a drainage area, the infiltration loss would be less than if the same volume of rainfall was uniformly distributed over the basin, other factors being the same.

(*b*) The infiltration capacity of a given soil tends to decrease as the duration of rainfall-excess increases. Consequently, the infiltration loss resulting from two successive periods of rainfall in the same portion of a basin would be less than if the same volumes of precipitation had occurred in different portions of the drainage area.

(*c*) The position of rainfall centers during successive periods of a storm may have a very great effect on the regimen of runoff from a given volume of rainfall-excess, particularly in large drainage basins in which a wide range of variation in the location of rainfall-excess volumes during successive periods of a design storm is possible.

Studies of a large number of major storms have revealed that intense rainfall covering areas of several thousand square miles may occur in a wide variety of patterns and that the maximum intensities may occur near the beginning, middle, or end of the precipitation period. Mountain ranges less than a few thousand feet in height affect the frequency of occurrence of certain rainfall patterns but apparently do not preclude the occurrence of patterns radically different from the normal during certain unusual storm conditions. The normal direction of movement of air masses over the central United States has caused most of the isohyetal patterns to be oriented in approximately a southwest-northeast direction, but the patterns of several important storms covering several thousand square miles have occurred with their major axes at right angles to the normal direction. Apparently, a considerable range in assumptions regarding rainfall patterns and intensity variations can be made in devel-

oping design-storm criteria for relatively small basins, without their being inconsistent with meteorological causes.

A high degree of conservatism is required in preparing design-storm rainfall estimates to be used in connection with the determination of spillway capacities for important dams. Any increase in rainfall quantities, and/or modifications in the pattern or sequence of rainfall increments, that appear to be necessary to assure a conservative estimate of the critical design storm should be included in the adopted estimate.

Three general methods of developing quantitative design-storm estimates are in common use:

Method 1. Computation of a maximum rainfall depth-duration relation for the size of area involved, based on rainfall data for a large number of storms that are considered possible of occurrence in the given region, and the development of a hyetograph to represent the critical sequence of rainfall quantities corresponding to the adopted depth-duration curve.

Method 2. Transposition of the isohyetal pattern of an actual storm to a critical position over the given drainage basin, without major changes in pattern or chronology of rainfall increments.

Method 3. Modified transposition method, in which it is assumed that the direction and/or rate of translation of rainfall zones during the record storm might have differed in such a manner as to have resulted in a more critical sequence and concentration of rainfall increments over an area comparable to the basin under study, the modifications assumed being predicated on meteorological studies.

Method 1 is most directly applicable in deriving spillway design-storm estimates for basins less than a few thousand square miles in area. The procedure may be used in preparing similar estimates for larger basins but greater approximations are required in estimating the volume and areal distribution of rainfall in successive intervals of the storm. Method 2 is applicable to all-size areas, but it is most useful in studies pertaining to basins having an area greater than a few thousand square miles, in which variations in intensity and areal distribution of rainfall during successive intervals of the storm have major effects on infiltration losses and the concentration of runoff. The use of the method is limited to studies in which data are available for a storm of record that is considered suitable, possibly with minor modifications, as design-storm criteria. Method 3 is usually required only in studies pertaining to large drainage basins.

37. Method 1. Maximum Rainfall Depth-Duration Data, and Rainfall-Excess. In preparing estimates of maximum probable-rainfall rates for basins less than a few thousand square miles in area, it is usually reasonable to assume that rainfall quantities equal to the maximum average depths observed over a given-size area in various storms of record in a particular region may eventually occur over any equal area within it, provided there is no appreciable difference in topography between the respective points, or the basin is not extremely irregular in shape. Inasmuch as the isohyetal patterns of major storms seldom

conform exactly to the shape of a drainage basin, rainfall values taken from depth-area curves are usually 10 to 15 per cent higher than would be obtained by transposition of the storms to the specific basin. This fact may be taken into account in selecting final margins of safety. The maximum average depths of rainfall over an area corresponding to the basin under study may be computed for various periods of time, using mass-rainfall curves developed in the manner outlined in Art. 12 as a basis for determining the quantities of rainfall in the respective periods (Art. 65, Ref. 17). Examples of maximum rainfall

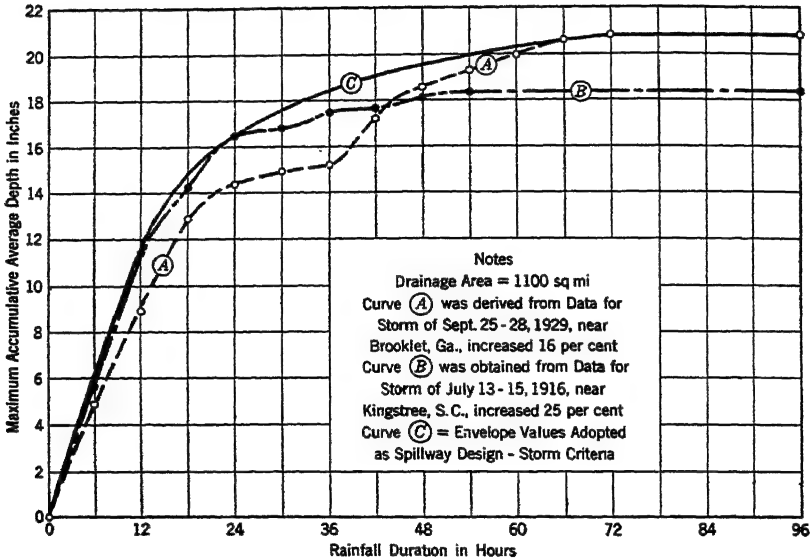


FIG. 16. Maximum rainfall depth-duration curves.

depth-duration curves are shown in Fig. 16. Curves A and B were based on maximum depth-duration data for two major storms and are intended to include allowances referred to in Art. 35, subparagraphs *b* and *c*. Curve C was drawn to envelop all values represented by curves A and B.

The rainfall depth-duration relation required to produce the maximum flood discharge in a drainage basin is partially determined by the amount of natural and artificial storage capacity available for modulating the rate of runoff from intense rainfall.

In a drainage basin having relatively small valley storage capacities, the peak rate of runoff from rainfall corresponding to curve B of Fig. 16 may exceed the peak rate that would result from the greater volume but less intense rainfall represented by curve A. In basins characterized by large valley storage capacities, the reverse might be true. The creation of an artificial reservoir in a natural drainage basin may either increase or decrease the storage capacity that would affect flood runoff rates immediately below the dam, the amount of change depending to an important extent upon the type and size of spillway

adopted. Inasmuch as an enveloping rainfall depth-duration curve represents the greatest volume and highest intensities represented in any of the basic storms, a design-storm estimate based thereon will assure a more conservative flood-discharge estimate than any one of the basic storms, regardless of storage-discharge characteristics of the natural basin or artificial reservoirs.

Although hypothetical hydrographs derived from rainfall estimates based on an enveloping depth-duration curve are somewhat more conservative than would be obtained from a depth-duration curve for any individual storm, the difference may be small. In basins characterized by rapid concentrations of flood runoff and small valley storage capacities, the critical flood would result from intense rainfall of relatively short duration. In such storms, runoff from rainfall of longer duration would not add appreciably to the flood peak but would simply prolong the flood. On the other hand, in basins characterized by large natural or artificial storage capacities, the high rate of runoff from short-duration, high-intensity rainfall would be modulated by storage to a large extent, and the effect of the longer-duration rainfall would be of greater importance. For use in computing hypothetical hydrographs, rainfall quantities for convenient unit periods of time are scaled from the adopted enveloping depth-duration curve. A unit period of 6 hr is a convenient standard for areas larger than approximately 100 sq mi. The sequence of rainfall increments necessary to give the critical rate of runoff can be arbitrarily determined by trial applications of the unit hydrograph to various groupings of the increments, inasmuch as it has been shown by records that the maximum rainfall intensities can occur near the beginning, middle, or end of the precipitation period. A typical arrangement of rainfall increments is shown in the hyetograph of Fig. 17.

The following procedure may be used to obtain an estimate of the maximum probable volume of rainfall-excess to be expected from a design storm resulting in rainfall quantities corresponding to the adopted enveloping depth-duration curve.

(*a*) The minimum initial loss and infiltration index values to be expected in the basin under study during the season in which the design storm is likely to occur are estimated by analyses of hydrologic records for the particular basin. (See Art. 16.)

(*b*) Rainfall quantities during the first periods of the design storm are assumed to be lost to runoff until the accumulative rainfall is equal to the initial loss, and thereafter a uniform rate of loss equal to the minimum infiltration index is assumed. The computations are illustrated by data tabulated beneath the hyetographs in Fig. 17.

In estimating rainfall-excess quantities in the manner outlined above it is assumed that rainfall-excess during successive periods of the design storm would have approximately the same areal distribution as occurred during the storm considered in deriving the infiltration index. If such an assumption cannot be made without the risk of serious error, either Method 2 or 3 should be followed in preparing the design-storm estimate, in order that the effects of

probable variations in rainfall continuity and areal distribution can be more closely approximated. The probability of significant errors in estimating in-

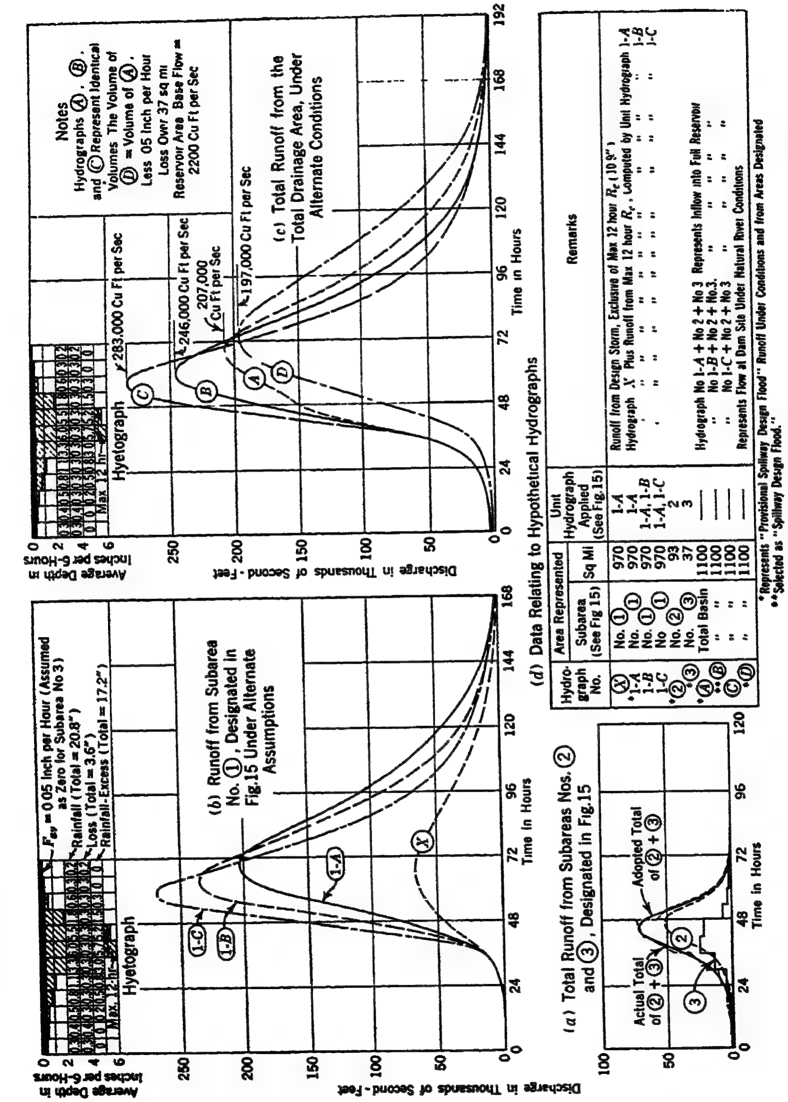


FIG. 17. Hypothetical hydrographs of runoff from spillway design storm.

filtration losses normally increases with an increase in the size of drainage area involved and with increase in infiltration capacities in the basin.

38. Method 2. Transposition of Record Storm: Rainfall-Excess Estimates. Information available for an actual storm of record may form a reasonable

basis for the spillway design-storm estimate, both with regard to rainfall quantities and intensity variations. The data may be utilized conveniently in the following manner:

(*a*) Superimpose an outline of the given drainage basin on the total-storm isohyetal map of the selected record storm in such manner as to place the highest rainfall quantities in a position that would result in the maximum flood runoff.

(*b*) Construct a network of Thiessen polygons for precipitation stations in and near the basin in its transposed position, prepare mass-rainfall curves for the respective stations, and complete the rainfall analysis in exactly the manner followed in computing infiltration indices from actual floods. (See Table 4, Part I.)

(*c*) Increase or decrease the observed rainfall values as considered necessary to assure an adequate design-storm rainfall estimate for the purpose involved.

(*d*) On the basis of data obtained by analyses of actual floods in the given basin and similar areas, select minimum values of initial loss and infiltration indices considered applicable to the design storm, and estimate corresponding rainfall-excess quantities in the manner followed in deriving infiltration indices. (See Table 4, Part II.)

(*e*) Plot rainfall and rainfall-excess data for representative zones in the manner illustrated in Fig. 8. If tributary flood flows are to be combined by flood-routing methods, the zones selected may conveniently correspond to the respective tributary basins.

39. Method 3. Modified Storm Transpositions: Rainfall-Excess Estimates. The problems involved in the determination of critical design-storm estimates for very large drainage areas differ somewhat from those encountered in preparing similar estimates for small basins. As a general rule, the critical floods in small basins result primarily from extremely intense small-area storms of relatively short duration, whereas the greatest floods in large basins usually result from a series of less intense, large-area storms. Not only is it necessary to compute maximum volumes of rainfall to be expected over large basins in various periods of time, but the most critical distributions and locations of rainfall quantities that are considered reasonably probable during successive periods of the storm must be estimated. (See Art. 36.)

Comprehensive meteorological studies similar to those outlined in Art. 35 are particularly important as a basis for determining spillway design-storm estimates for large drainage basins that are both reasonable and conservative. When approximate estimates are necessary, a procedure similar to the following probably assures the most reliable results:

(*a*) Examine available reports by Federal, State and private agencies that contain information regarding major storms of record within several hundred miles of the project under study, and select for investigation those storms that appear to have been capable of producing major floods over areas equal to the given drainage basin. (See Art. 65, Refs. 13–17.) In making such selections, the areal distribution and intensity of rainfall, and conditions affecting infiltra-

tion losses, or contributions from melting snow coincident with the storm periods as well as the volume of rainfall, should be taken into consideration.

(*b*) Make such preliminary compilations of data as are required to determine more definitely which of the tentatively selected storms offer the most suitable basis for a design-storm estimate for the given project.

(*c*) Obtain all isohyetal maps, precipitation records, mass-rainfall curves, and duration-depth-area data available for the storms selected for final study. If necessary, supplement these data by preparing isohyetal maps and mass-rainfall curves for each distinct period of heavy rainfall, covering storm areas appreciably larger than the basin under study.

(*d*) Review available information regarding the meteorological conditions resulting in the respective storms, to determine whether it is reasonable to assume that the movement of the zones of heaviest precipitation during successive distinct periods of a storm series might have been such as to cause a greater accumulation and/or a more critical concentration over an area comparable to the basin under study than actually occurred in the record storm. In this connection, a study of the rainfall patterns and the movement of rainfall centers during several major storms of record in the region may serve as a reasonable basis of judgment.

(*e*) Superimpose the outline of the given drainage basin on the isohyetal patterns that represent the successive rainfall periods of a particular storm, in positions corresponding to the movement of rainfall centers assumed in step *d*. It is not necessary that the orientation of the basin outline be the same on isohyetal maps representing successive periods of rainfall that are separated by intervals of several hours, but the orientations should be reasonably consistent with the assumptions regarding the meteorological causes of the storm.

(*f*) Complete the computation of rainfall and rainfall-excess quantities for each transposition in the manner outlined in Art. 38 subparagraphs *b* to *e*.

(*g*) Compare the rainfall-excess quantities computed for the various major storms considered to determine the critical rainfall series to be adopted as the spillway design storm for the given project.

40. Flood-Flow Contributions from Melting Snow. In many drainage basins the maximum runoff from snow-melt occurs during the spring or late winter storm season. In preparing design-storm estimates for these basins, it is necessary to consider the possibility of snow-melt augmenting the runoff from heavy rainfall.

The rate and quantity of runoff from melting snow are determined primarily by the water equivalent of the snow cover, ground conditions affecting infiltration losses, and the rate of release by melting. The factors affecting runoff from a snow cover not only vary by geographical regions but also by seasons, and from period to period in the same area.

Because of the many variable and complex factors that influence the rate and quantity of runoff from a snow mantle, and the indefinite conclusions now prevailing regarding the processes involved, it is impracticable to present a

satisfactory generalized procedure for estimating the critical contributions by snow-melt during design floods. It is the intention in the following paragraphs to give a general impression of the problems involved, and to suggest methods by which quantitative estimates of runoff from snow-melt can be prepared for certain of the less complicated situations. The methods are considered applicable to basins in which conditions affecting runoff from snow-melt can either be treated as approximately uniform, or the assumption can be made that such lack of homogeneity as exists will be reflected in empirical coefficients derived from hydrologic records for the basin. The methods are considered most directly applicable to moderate-size drainage basins less than a few thousand feet in elevation. For discussions and information pertaining to the computation of snow-melt quantities for more complicated conditions, the reader is referred to reports and papers on the subject listed in the accompanying bibliography, and to various technical publications. Particular attention is invited to the "Transactions of the American Geophysical Union" and to sources of information listed therein.

41. Ground Conditions. The infiltration capacity of the soil beneath a snow blanket has a very important influence on the quantity of runoff from snow-melt. If the ground is frozen, the infiltration loss may be very small, particularly if the soil is of reasonably fine texture and the field moisture content is high at the time of freezing. The infiltration capacity of a coarse-textured soil containing very little capillary water may not be appreciably reduced by freezing.

Inasmuch as snow is a poor conductor of heat, very little additional freezing of moisture in the soil may be expected after a moderate depth of snow has been accumulated. Therefore, the optimum ground conditions for high volumes of runoff from snow-melt result when a severe freeze occurs prior to the occurrence of deep snow, and the capillary field moisture is high at the time of the freeze.

42. Water Equivalent of Snow Cover. The volume of runoff from a snow cover is limited to the amount of water contained in the form of snow and ice, plus the water stored in the snow structure. However, it should be observed that the water equivalent of a snow cover may substantially exceed the moisture equivalent of precipitation that occurs in the form of snow, by reason of the retention of rainfall in the voids of the snow structure, either by capillarity or by the formation of ice.

Under average conditions, 10 in. of freshly fallen snow represents approximately 1 inch of water. However, as the snow remains on the ground the density increases by reason of packing, by the possible addition of moisture from rainfall and condensation, and by an alternate thawing and freezing process. The density of the snow occasionally is as high as 50 to 60 per cent, and it may be even higher in extreme cases. Densities of 20 to 30 per cent are most common.

43. Free Water in Snow Cover. An appreciable amount of moisture may be retained in a snow cover in a "free" or unfrozen state. The amount of free

water that can be retained in a snow mantle varies with the texture and general characteristics of the snow structure. A fairly comprehensive series of field studies have indicated that, under average conditions, a column of snow is capable of retaining free water equal to approximately 20 to 25 per cent of the "dry" weight of the snow (Art. 65, Ref. 18).

A snow cover is said to "ripen" as its wetness is increased by melting and/or absorption of rainfall that is held in a liquid state in the snow structure. In order that the maximum probable rate of contribution of runoff from a snow cover may coincide with a major winter storm, a period of ripening must precede the heaviest rainfall. Inasmuch as major winter storms are frequently preceded by periods of light rainfall, in estimating the maximum probable winter flood it is usually reasonable to assume that the snow mantle contains an appreciable quantity of unfrozen water which will be released as the snow melts, thus augmenting the runoff from snow actually melted by heat transfers during the rainfall period.

44. Rate of Release by Melting—General. In addition to the infiltration characteristics of the underlying soil, the following factors have major influences on the rate of contribution of runoff from a snow cover during flood periods:

(a) Degree of "ripening" by antecedent heat transfers and absorption of rainfall. (See Art. 43.)

(b) Transfer of heat from the air, as the result of the temperature difference between the air and the snow surface, and condensation of moisture on the snow surface.

Under ordinary circumstances there is an appreciable exchange of heat by means of radiation between a snow cover and space. The snow absorbs solar radiation and at the same time radiates heat back into space. During storm periods, heavy cloud covers reduce the amount of solar radiation reaching the snow surface, but radiation of heat from the cloud cover to the snow tends to offset the reduction of insolation to some extent. The net loss or gain of heat to the snow mantle by radiation during storm periods has not been quantitatively determined. For lack of a better method, it is assumed in the following paragraphs that the effects of radiation during storm periods will be reflected in empirical coefficients derived from hydrologic records for the drainage area under study (see Art. 45).

45. Heat Transfers from Air. Studies made by the Hydrometeorological Section of the Weather Bureau in cooperation with the Corps of Engineers, U. S. Army (Art. 65, Refs. 13, 14) indicate that melting of snow *during storm periods* is primarily the result of the heat transferred from the air to the snow mantle in two ways: (1) by direct heat exchange due to the temperature difference between the air and snow, and (2) by the release of heat through condensation of moisture on the snow surface. This transfer of heat results in conversion of the solid portion of the snow into liquid form at the rate of 1 g per 80 cal, the heat of fusion of ice. Inasmuch as the heat of fusion of ice is only 80 cal and the heat of vaporization of water is 600 cal per g, the moisture

condensed on the snow surface melts 7.5 times its own weight in snow (Art. 65, Ref. 14).

The thermodynamics involved in the melting of snow are discussed in detail by W. T. Wilson (Art. 65, Ref. 19) and Phillip Light (Art. 65, Ref. 20). Wilson presents the following formulas to represent the depth of snow melted in a period of 6 hr:

$$Dm = K_1V(e - 6.11) \tag{13}$$

$$Dc = K_2V(T - 32°) \tag{14}$$

$$\begin{aligned} Da &= Dm + Dc \\ &= K_1V(e - 6.11) + K_2V(T - 32) \end{aligned} \tag{15}$$

in which Dm is the water equivalent of snow melted in 6 hr by moisture condensation (plus the condensate) in inches depth; Dc is the depth melted in 6 hr by direct heat exchange from the air by convection and conduction; Da is the total water equivalent of snow melted in 6 hr by the processes referred to above; V is the wind velocity in miles per hour; T is the dry-bulb temperature in °F; e is vapor-pressure in millibars; K_1 is a coefficient involving the latent heat of ice, exposures of instruments, and conversion units; K_2 is a coefficient involving the latent heat of ice, exposure of instruments, conversion units, air density, and certain considerations involved in the theory of turbulence. The coefficients K_1 and K_2 are related, K_1 being equal to approximately 3 to 4 times the value of K_2. For the most common arrangement of instrument installations, and stations less than a few thousand feet in elevation above sea level, Eq. 15 may be written in the following approximate form:

$$Da = V(0.002T + 0.006e - 0.100) \tag{16}$$

There are a number of basin characteristics that determine the areal distribution and variations in depth of snow, the degree of ripening in various areas, and the rate of heat transfer during storm periods. The most important are range in elevations, topography, type of cover (forests, etc.) and orientation of the basin with respect to the direction of movement of warm air masses (Art. 65, Ref. 24). These factors can be evaluated approximately by rational analyses, but it is necessary to make certain arbitrary assumptions or utilize empirical factors (Art. 65, Ref. 13). In order to obtain an estimate of the contribution of runoff from the snow cover in a drainage area of moderate size that takes into account the characteristics of the snow cover, meteorological influences in the region and conditions peculiar to the particular basin, Eq. 16 should be modified by a coefficient, K, to read as follows:

$$Da = KV(0.002T + 0.006e - 0.100) \tag{17}$$

The constant K is a basin characteristic that should be evaluated empirically. If records are available for floods representing reasonably large quantities of runoff from snow-melt in conjunction with major storms, the constant

may be evaluated by substituting observed or deduced values of Da, V, T, and e in Eq. 17 and solving for K. The empirical value of K would then be used in Eq. 17 to estimate snow-melt quantities for other assumed conditions. Average values of K computed in this manner for drainage areas in the Ohio River Basin above Pittsburgh, Pa. (Art. 65, Ref. 13), ranged from 0.60 to 0.72. Similar values have been obtained for several New England basins. However, these values should not be assumed to be universally applicable.

It should be observed that Eqs. 13 to 17 were derived to indicate the rate of melting of snow from the various heat transfers and do not contain allowances for the release of free water accumulated in the snow cover prior to the storm, except insofar as such allowances may be represented in an empirically determined coefficient K. If K is computed from records of floods in which the runoff resulted from the occurrence of heavy rainfall on a ripened snow mantle, an allowance for the release of free water stored in the snow at the time of the storm is contained in the empirical factor. Otherwise, the computed K factor should be increased to allow for th. probability of a ripened snow condition in conjunction with the maximum probable winter storm. The proper amount of increase must be estimated on the basis of studies of records of winter floods and a consideration of the conditions that are reasonably likely to occur.

46. Melting by Rainfall. Although the quantity of snow melted by rainfall is usually small in comparison with quantities melted by processes enumerated above, the amount may be significant in some basins during unusual storms. The depth may be computed by the following formula:

$$Dr \doteq \frac{P(T - 32)}{144} \qquad [18]$$

in which Dr is the depth of snow-melt in inches, P is the precipitation in inches, and T is the wet-bulb temperature of the air, in °F.

47. Snow-Melt Estimates by Heat-Transfer Formulae. Estimates of runoff from snow-melt during critical winter storms over a drainage basin of moderate size may be prepared by use of Eqs. 16, 17, and 18 in approximately the following manner:

(*a*) For use in evaluating K in Eq. 17, select for analyses the most important winter floods of record that were characterized by large volumes of runoff from snow-melt in the drainage basin under study, or in similar basins in the region, and plot the hydrographs of observed runoff.

(*b*) Compute rainfall quantities, mean temperatures, average wind velocities, and vapor pressures for each 6-hr period of the storms selected in step *a*, and plot graphically, or tabulate the data above the observed runoff hydrograph in proper time relation.

(*c*) Deduct the estimated base flow from the observed hydrographs to obtain hydrographs representing runoff from snow-melt and rainfall combined.

(*d*) Using unit hydrographs derived from the analyses of floods unaffected by snow-melt, estimate by a trial-and-error procedure the hyetograph of

6-hr values of rainfall-plus-snow-melt required to reproduce the hydrographs computed in step *c*.

(*e*) Estimate infiltration losses for the flood periods being analyzed, using as a basis information on infiltration capacities obtained by analyses of floods that were unaffected by snow-melt, and taking into consideration all available data on the condition of the ground during the floods under study.

(*f*) From the synthetic hyetograph computed in step *d*, subtract the observed rainfall values, determined in step *b*, and add the infiltration loss computed in step *e*, to obtain estimates of water released from the snow cover during successive 6-hr periods of each storm.

(*g*) Substitute computed or observed values of Da, T, V, and e in Eq. 17 to determine values of K applicable to each 6-hr period of the record storms investigated, and average values for each storm period.

(*h*) From available records of snow conditions in the region, estimate the critical quantity, distribution, and degree of ripeness of snow likely to exist immediately before the maximum probable storm of the snow season.

(*i*) On the basis of data obtained by the study of major winter storms in the region, supplemented insofar as practicable by meteorological analyses, select values of T, V, and e to be used in estimating critical rates and volumes of snow-melt during the maximum probable winter storm, and compute theoretical values of snow-melt for successive 6-hr periods by Eqs. 16 and 18.

(*j*) Taking into consideration the characteristics of snow mantles that prevailed during the record floods analyzed in steps *a* to *g*, and conditions assumed to prevail during the maximum probable winter storm, select values of K to be used in estimating critical volumes of runoff from snow-melt during successive periods of the winter design storm. The following procedure is suggested:

1. Select an average value of K for the storm period, and compute the total quantity of snow-melt for the design-storm period by multiplying the total theoretical quantity computed in step *i* by this average K.
2. Assume that K will be zero until the snow has ripened sufficiently to permit outflow from the snow structure, and that it will increase progressively during the storm until a maximum materially exceeding the average value for the total storm period is reached, and thereafter will decrease somewhat as the supply is depleted in portions of the drainage area. In some instances, it may be desirable to assume that the snow is in a ripened condition at the beginning of the design storm, in which case the initial values of K may be relatively high. The sum of 6-hr quantities should equal the total amount obtained in step *j* 1.

48. Snow-Melt Estimates by Degree-Day Method. Several investigators have obtained approximate correlations between "degree-days" of temperature above approximately 32° F and the runoff from snow-melt expressed in inches depth from the drainage area. In the majority of the studies, the maximum snow-melt runoff ranged from 0.04 in. (Art. 65, Refs. 21, 22) to

0.09 in. (Art. 65, Ref. 23) per degree-day. However, in exceptional cases, the runoff from snow-melt has exceeded 0.22 in. per degree-day for at least 24 hr (Art. 65, Ref. 14). Although the range in values given above is very large, it is no greater than might be expected in consideration of the variables discussed in Arts. 40 to 45. Inasmuch as the mean daily temperature of the air is only one of several factors that have major influences on the contribution of runoff from a snow cover, the degree-day method is suitable only for approximate studies (Art. 65, Ref. 24).

49. Hyetograph of Snow-Melt Plus Rainfall. The estimated snow-melt quantities corresponding to successive unit periods of a design storm may be added directly to rainfall values for the respective periods to obtain the design-storm hyetograph. Hypothetical hydrographs of runoff from the combined rainfall-snow-melt hyetograph may be computed in the same manner as followed in developing hydrographs of runoff from rainfall-excess estimates alone.

G. Required Spillway Capacity

50. Hypothetical Hydrographs. Inasmuch as the critical volume and concentration of runoff into a reservoir cannot be accurately determined, it is desirable to estimate the extent of variations in computed maximum reservoir levels that would result from various changes in basic assumptions. The following series of computed hydrographs, derived to represent runoff from the spillway design storm rainfall-excess under various conditions, is useful in the determination of the spillway capacity required to provide safely for the most critical flood runoff from a particular basin:

(*a*) *A "Provisional Spillway Design Flood" hydrograph representing runoff from the area above the dam site under natural river conditions.* This hydrograph serves as a basis for comparing the design-flood criteria with maximum floods of record in natural river basins. (See Hydrograph D, Fig. 17*c*.)

(*b*) *A "Provisional Spillway Design Flood" inflow hydrograph representing runoff into a full reservoir.* This hydrograph may be developed in the manner outlined in the discussion of "Reservoir Inflow Unit Hydrographs" (Art. 34). The difference between the provisional spillway design-flood hydrographs for natural river conditions above the dam site and inflow into the reservoir, respectively, reflect the effect of the reservoir in modifying the regimen of runoff. (See Hydrograph A, Fig. 17*c*.)

(*c*) *A group of hypothetical hydrographs representing greater concentrations of runoff but having the same volume as the provisional spillway design-flood inflow hydrograph.* These arbitrarily modified hydrographs are used to determine the amount of increase in maximum reservoir level that would result from possible variations in the concentration of runoff and are of value in estimating the factor of safety inherent in the freeboard storage of a reservoir. (See Hydrographs B and C of Fig. 17*c*.)

(*d*) *"A Spillway Design Flood" inflow hydrograph, or the hydrograph finally accepted as representing the critical volume and concentration of flood runoff into*

the reservoir under the most extreme conditions considered reasonably possible. The spillway design-flood inflow hydrograph is assumed to reflect all factors of safety necessary to assure a safe estimate of the maximum reservoir level that would be attained with the adopted spillway capacity and method of operation.

The provisional spillway design-flood inflow hydrograph is derived to represent critical rates of runoff into the reservoir from the spillway design storm, and for some reservoirs may be considered sufficiently conservative to serve as the final spillway design-flood criteria. For other reservoirs, a general review of the adequacy of basic data and reliability of analysis may justify certain modifications in the provisional spillway design-flood inflow hydrograph in order to assure a safe spillway design-flood estimate.

51. Outline of Hydrograph Computations. The steps followed in computing hypothetical hydrographs for use in estimating spillway requirements for reservoirs in drainage basins less than a few thousand square miles in area are illustrated by computations pertaining to a reservoir project in the Saluda River Basin, S. C.

(*a*) The hydrographs and related data for six major flood rises and three minor rises were analyzed to determine minimum infiltration indices for the basin and to obtain unit hydrographs corresponding to representative rainfall-excess distributions. (See Tables 4, 5, Figs. 6, 7, 8, and 13, and related discussions.)

(*b*) The relative magnitude and the meteorological characteristics of several major storms in the region were investigated to determine maximum probable rainfall quantities for the 1100 sq mi drainage area involved. Maximum rainfall depth-duration curve C of Fig. 16 was selected as the design-storm criteria. Six-hr rainfall values of the design storm are shown in the hyetographs in Fig. 17.

(*c*) An initial loss of 0.4 in. and an infiltration index of 0.05 in. per hr were adopted as representative of minimum values likely to prevail during the spillway design flood. The computed design-storm rainfall-excess quantities are indicated in the hyetographs in Fig. 17.

(*d*) Unit hydrograph No. 5 of Fig. 15*a* was derived to represent runoff from unit rainfall quantities above the dam site under natural river conditions, assuming rainfall distributions comparable to those recorded during the Oct. 1–7, 1929, flood rise (Fig. 8). The "Provisional Spillway Design Flood" hydrograph for natural river conditions above the dam site was computed by application of unit hydrograph No. 5, Fig. 15*a*, to the spillway design-storm rainfall-excess quantities. (See Table 9.)

(*e*) The rate of inflow into a full reservoir from subareas Nos. 2 and 3 shown in Fig. 15*c* was computed by application of unit hydrographs Nos. 2 and 3 of Fig. 15*b*, respectively, to the entire series of 6-hr rainfall-excess quantities of the design storm. The individual hydrographs and the adopted total for the two subareas are shown in Fig. 17*a*. Because of the relatively small runoff volumes involved, the "Adopted-Total" hydrograph shown in

TABLE 9

COMPUTATION OF HYPOTHETICAL HYDROGRAPH *

Time in Hours	12-Hour Unit Hydrograph in sec-ft (No. 5, Fig. 15)	Rainfall-Excess in Inches per 12 Hr	Surface Runoff from Rainfall-Excess Units, in Second-Feet					Base Flow in sec-ft	Total Discharge in sec-ft
			Rainfall-Excess, in Inches				Subtotal		
			0.7″	3.8″	10.9″	1.8″			
1	2	3	4	5	6	7	8	9	10
6	800	0							
12	2,900								
18	5,900	0.7	560				560	2,200	2,760
24	8,600		2,030				2,030	2,200	4,230
30	10,500	3.8	4,130	3,040			7,170	2,200	9,370
36	11,600		6,020	10,200			16,220	2,200	18,400
42	11,800	10.9	7,350	22,400	8,720		38,470	2,200	40,700
48	11,300		8,120	32,700	31,600		72,420	2,200	74,600
54	10,100	1.8	8,260	39,900	64,300	1,440	113,900	2,200	116,000
60	8,800		7,910	44,100	93,700	5,220	150,930	2,200	153,000
66	7,600		7,070	44,800	114,400	10,600	176,870	2,200	179,000
72	6,500		6,160	42,900	126,400	15,500	190,960	2,200	193,000
78	5,500		5,320	38,400	128,600	18,900	191,220	2,200	193,000
84	4,500		4,550	33,400	123,200	20,900	182,050	2,200	184,000
90	3,500		3,850	28,900	110,100	21,200	164,050	2,200	166,000
96	2,700		3,150	24,700	95,900	20,300	144,050	2,200	146,000
102	1,960		2,450	20,900	82,800	18,200	124,350	2,200	127,000
108	1,330		1,890	17,100	70,800	15,800	105,590	2,200	108,000
114	940		1,370	13,300	60,000	13,700	88,370	2,200	90,600
120	630		930	10,300	49,000	11,700	71,930	2,200	74,100
126	430		660	7,450	38,200	9,900	56,210	2,200	58,400
132	250		440	5,050	29,400	8,100	42,990	2,200	45,200
138	130		300	3,570	21,400	6,300	31,570	2,200	33,800
144	50		180	2,390	14,500	4,860	21,930	2,200	24,100
150			90	1,630	10,200	3,530	15,450	2,200	17,700
156			40	950	6,870	2,390	10,250	2,200	12,500
162				490	4,690	1;690	6,870	2,200	9,070
168				190	2,730	1,130	4,050	2,200	6,250
174					1,420	770	2,190	2,200	4,390
180					550	450	1,000	2,200	3,200
186						230	230	2,200	2,430
192						90	90	2,200	2,290

* Drainage area = 1100 sq mi: All discharges are instantaneous values at end of hour given in Col. 1.

Fig. 17*a* was assumed to represent critical runoff from subareas Nos. 2 and 3 during the design storm, regardless of modifications in assumptions pertaining to other portions of the drainage basin.

(*f*) Hydrograph X was computed by applying unit hydrograph No. 1-A, Fig. 15*b*, to successive 6-hr rainfall-excess increments of the design storm, with exception of the two maximum 6-hr values, which were omitted. Hydro-

graph X was assumed to represent the critical rate of runoff from all rainfall-excess increments of the design storm over subarea No. 1, Fig. 15*c*, other than the two maximum 6-hr quantities.

(*g*) Hydrographs Nos. 1-A, 1-B, and 1-C of Fig. 17*b* were computed by applying unit hydrographs Nos. 1-A, 1-B, and 1-C of Fig. 15*b* to the two maximum 6-hr rainfall-excess values of the design storm, and adding the respective partial hydrographs obtained thereby to hydrograph X. Hydrograph 1-A of Fig. 17*b* was chosen to represent the "Provisional Spillway Design Flood" runoff for subarea No. 1 of Fig. 15*c*, and hydrographs Nos. 1-B and 1-C were used to reflect more severe concentrations of runoff from the most intense 12-hr period of the design storm, with no change in assumptions regarding other periods of the design storm.

(*h*) On the basis of data obtained from studies of floods of record in the basin, the base flow during the design storm was estimated as 2 sec-ft per sq mi, or 2200 sec-ft from the entire drainage area.

(*i*) Hydrographs Nos. A, B, and C of Fig. 17*c* were computed by adding hydrographs Nos. 1-A, 1-B, and 1-C, alternately, to the "Adopted-Total" hydrograph for subareas Nos. 2 and 3 (Fig. 17*a*) and the assumed base flow of 2200 sec-ft, taking into consideration the proper time relation of each hydrograph to the design-storm values.

52. Reservoir Flood-Routing Computations. The process of computing the reservoir stage, storage volumes, and outflow rates corresponding to a particular hydrograph of inflow is commonly referred to as "flood routing." The maximum reservoir level obtained by routing a particular flood through a reservoir is determined by the following:

(*a*) Initial reservoir stage.
(*b*) Rate and volume of inflow into the reservoir.
(*c*) Rate of outflow:
  1. Discharge of regulating outlets, power penstocks, etc.
  2. Discharge over spillway.
(*d*) Storage capacity above initial reservoir level per unit increase in stage.

53. Reservoir Stage at Beginning of Floods. In estimating spillway requirements for reservoirs located in drainage basins in which volumes of flood runoff are normally high in comparison with the storage capacity of the proposed reservoir, it is usually necessary to assume that the reservoir would be filled to the normal maximum pool level at the beginning of the spillway design flood. Even though the plan of reservoir operation contemplated may indicate that a portion of the storage capacity below the normal maximum pool level probably would be available at the beginning of the spillway design flood, the possibility of improper operation of regulating outlets as the result of incorrect flood predictions, mechanical difficulties, plugging of conduits by debris, or negligent attendance may justify the assumption of a full reservoir at the beginning of the design flood. Moreover, future developments may require revisions in the original plan of operation, or changes in the use of the

reservoir that would increase the probability of a full reservoir at the beginning of the spillway design flood.

If the capacity of a reservoir is large in proportion to the runoff volumes during major floods in the basin, studies should be made to determine as accurately as possible the maximum reservoir elevation that might reasonably be expected to prevail at the beginning of the spillway design flood. Hypothetical reservoir operation studies should be made to determine the maximum pool elevations that would have occurred under the proposed plan of operation if the reservoir had been in existence during the period of stream-flow records. If the stream-flow records cover a period of sufficient length to reflect average flood conditions, frequency curves may be computed to indicate the probability of various reservoir levels under the hypothetical operation plan. Although the probability of any particular reservoir level coinciding with the occurrence of the spillway design flood is indeterminate, the pool-level frequency data based on records of past floods may serve as a useful guide in making reasonable assumptions regarding the pool level at the beginning of the spillway design flood. In connection with the determination of spillway requirements for several important flood-control dams in the United States, it has been assumed that the reservoir level at the beginning of the spillway design flood would equal the elevation attained by a flood having an average frequency of occurrence of once in 25 years under the assumed plan of operation.

54. Discharge Through Regulating Outlets. Under normal conditions, water may be released through regulating conduits during flood periods, in addition to flow over the spillway. However, it is generally considered good practice to assume that regulating outlets would be inoperative during the spillway design flood because of mechanical failures, blocking of the inlets by debris, or lack of attendance during the emergency. (See Art. 53.)

55. Reservoir Flood-Routing Method. Although there are definite relations between reservoir inflow, storage, and outflow, these relations are usually difficult to express algebraically. Consequently, a step-by-step computation procedure is ordinarily followed, whereby the increase or decrease in storage and rate of outflow resulting from the volumes of inflow during successive short increments of time are computed. Increments of inflow are computed for time periods sufficiently short to warrant the assumption that the mean of inflow rates and the mean of outflow rates at the beginning and end of the intervals would closely approximate the average rates for the respective periods.

One of the most widely used methods of routing floods through reservoirs involves the use of "Inflow-Storage-Discharge" (I.S.D.) curves (Art. 65, Ref. 25) similar to those shown in Fig. 18. A convenient method of developing I.S.D. curves is illustrated by the procedure followed in preparing Fig. 18 for use in routing the hydrographs shown in Fig. 17*c*. Two inflow intervals, 6 and 12 hr, were selected for alternate use. The 12-hr inflow interval was used in computations pertaining to periods of the floods during which the rates of change in inflow, storage, and outflow were relatively uniform, and

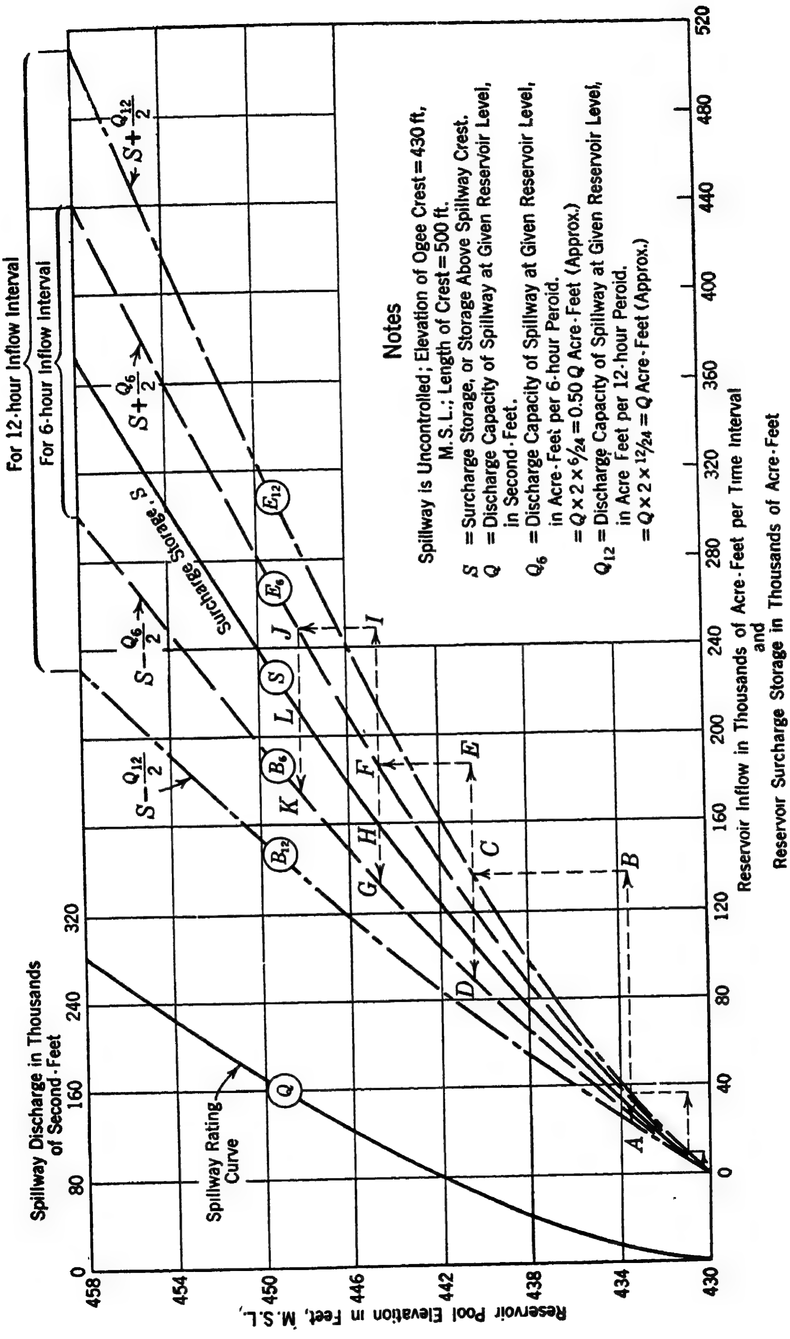


FIG. 18. Inflow-storage-discharge (I.S.D.) curves.

the 6-hr interval was used to evaluate the more rapid rates of change near the peak of the floods. The I.S.D. curves in Fig. 18 were constructed in the following manner:

(*a*) The storage-capacity curve for the reservoir above the spillway crest elevation was plotted as curve S, with acre-feet as abscissas and reservoir elevations as ordinates.

(*b*) The spillway rating curve was plotted as curve Q, with second-feet as abscissas and reservoir elevations as ordinates.

(*c*) Curve B_{12} was computed by subtracting from curve S one-half of the spillway discharge capacity at corresponding reservoir elevations, expressed in acre-feet per 12 hr, and curve E_{12} was computed by adding one-half the spillway discharge capacity in acre-feet per 12 hr to curve S. Inasmuch as 1 sec-ft is equal to 2 acre-ft per day (approximately), the abscissas of curve Q in second-feet are equal to the spillway discharge capacity in acre-feet per 12 hr. It may also be observed that the abscissa scale of curve S, in acre-feet, is double the abscissa scale of curve Q, in second-feet; therefore, points on curve B_{12} may be computed graphically by laying off distances to the left of curve S equal to the abscissas of curve Q for corresponding reservoir elevations, and curve E_{12} may be computed by laying off equal distances to the right of curve S. Curves B_6 and E_6 may be computed in a similar manner, taking into consideration the difference in the time intervals.

56. Sample Flood-Routing Computations. One convenient method of routing flood-flows through a reservoir by use of I.S.D. curves is illustrated by computations shown in Table 10. Cols. 3 to 5 contain data on inflow rates and volumes corresponding to hydrograph B of Fig. 17*c*. The first four inflow increments (Col. 5) were computed for 12-hr periods; therefore curves B_{12} and E_{12} of Fig. 18 were used in computations pertaining to the first 48 hr of the flood. For the period from 48 to 84 hr from the beginning of the flood, curves B_6 and E_6 were used with 6-hr runoff data, in order to obtain a more accurate definition of outflow and stage hydrographs.

An initial reservoir elevation of 430.0 ft was assumed at the beginning of the flood (see Art. 53). The routing procedure is illustrated graphically in Fig. 18. The first steps in the process are obscure because of the small volumes of inflow involved, but the procedure is similar to that indicated by the series A, B, C, pertaining to the fourth step of the routing. The 12-hr increment of inflow (112,700 acre-ft) was laid off from curve B_{12} as abscissa AB, and a vertical line was drawn from B to an intersection with curve E_{12} at C, to obtain the reservoir elevation (440.5 ft) at the end of the 48th hr of the flood. Inasmuch as the fifth increment of inflow was for a 6-hr period, the inflow value (98,600 acre-ft) was laid off from curve B_6 as abscissa DE, and the vertical line was projected to curve E_6 to obtain the reservoir level at the end of the 54th hr of the flood (444.6 ft). In this manner the entire reservoir-stage hydrograph tabulated in Col. 6 of Table 10 was computed. The rates of outflow (Col. 7, Table 10) corresponding to various reservoir levels were read from curve Q. The results are plotted in Fig. 20.

TABLE 10

SAMPLE FLOOD-ROUTING COMPUTATIONS

Time from Beginning of Flood, in Hours	Length of Interval (T), in Hours	Instantaneous Rate of Inflow into Reservoir (I), in c.f.s.[a]	Sum of Discharges, at Beginning and End of Interval ($I_1 + I_2$)	Volume of Inflow into Reservoir During Interval, in Acre-Feet[b]	Reservoir Elevation at End of Interval, in Feet, m.s.l.[c]	Spillway Discharge-Rate at End of Interval, in c.f.s.[c]
1	2	3	4	5	6	7
0		2,000			430.0	0
12	12	3,800	5,800	2,900	430.3	500
24	12	10,400	14,200	7,100	431.0	2,000
36	12	52,400	62,800	31,400	433.6	12,000
48	12	173,100	225,500	112,700	440.5	64,000
54	6	221,400	394,500	98,600	444.6	106,000
60	6	245,800	467,200	116,800	448.2	147,000
66	6	237,500	483,300	120,800	450.6	178,000
72	6	211,800	449,300	112,300	451.8	193,000
78	6	180,000	391,800	97,900	451.8	193,000
84	6	148,600	328,600	82,100	451.1	184,000
96	12	92,800	241,400	120,700	448.1	146,000
108	12	54,100	146,900	73,400	444.5	105,000
120	12	29,700	83,800	41,900	441.0	69,000
132	12	16,500	46,200	23,100	438.2	44,000
144	12	8,800	25,300	12,600	436.1	28,000
156	12	4,000	12,800	6,400	434.5	17,000
168	12	2,500	6,500	3,250	433.4	11,000
180	12	2,000	4,500	2,250	432.6	8,000
192	12	2,000	4,000	2,000	432.0	5,000

[a] Rate of inflow equals hydrograph B of Fig. 17c.
[b] Average rate of inflow during interval = $\frac{1}{2}(I_1 + I_2)$ (approximate).
If T = 12 hr, volume of inflow = $\frac{1}{2}(I_1 + I_2)$ (approximate).
If T = 6 hr, volume of inflow = $\frac{1}{4}(I_1 + I_2)$ (approximate).
[c] Computed by curves shown in Fig. 18.

The validity of the routing procedure outlined above may be proved in the following manner:

Let the line GI (Fig. 18) represent the volume of inflow in 6 hr.

GF = Rate of outflow at the beginning of the 6-hr period.

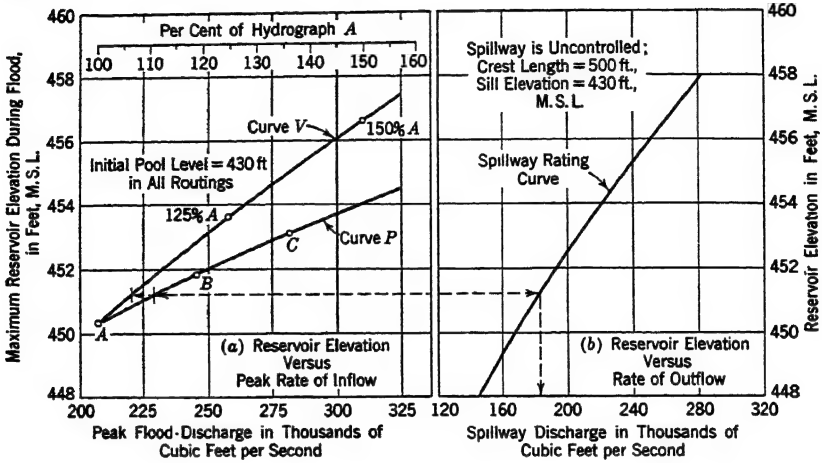
KJ = Rate of outflow at the end of the 6-hr period.

$(GF + KJ)/2$ = Mean rate of outflow.

$= GH + LJ$

$GI - (GH + LJ)$ = Volume of storage corresponding to difference in reservoir elevation (IJ) at beginning and end of 6-hr inflow period.

57. Graphical Presentation of Flood-Routing Results. The maximum pool elevation obtained by routing a particular flood hydrograph through a reservoir reflects the integrated effect of volume and rate of flood runoff, surcharge storage, and rate of outflow. Concise graphical presentations of the results obtained by routing hypothetical hydrographs similar to those listed in Art. 50 serve as valuable aids in studying the effect of various changes in assumptions regarding rates and volume of runoff.



Notes

Points A, B, and C on Curve P were obtained by Routing Hydrographs (A), (B), and (C), Respectively, of Fig. 17 (*c*), which Represent Identical Volumes of Runoff.

Curve V was Computed by Routing Hydrographs obtained by Increasing All Ordinates of Hydrograph (A) Fig. 17 (*c*) Various Percentages.

FIG. 19. Results of flood routings.

The form of charts illustrated in Fig. 19 have proved to be particularly useful. Point A in Fig. 19*a* represents the maximum reservoir elevation that would be obtained during the "provisional spillway design flood," under the plan of operation assumed in the sample computations outlined in Art. 56. Points on curve P were computed by routing hypothetical hydrographs representing the same total volume as the provisional spillway design flood but higher peak rates and greater concentrations of runoff. The quantitative effect of possible errors in estimating the critical regimen of runoff from the adopted spillway design storm may be determined from curve P. As a contrast, curve V was computed by routing hypothetical hydrographs representing various direct percentage increases in all ordinates of the provisional spillway design-flood hydrograph. The percentage increase in hydrograph ordinates is equivalent to an increase in *volume* of the design-storm rainfall-excess quantities, without change in the assumed regimen of runoff as reflected by

unit hydrographs used in developing the provisional spillway design-flood estimate. A comparison of the slopes of curves V and P serves as an index to the relative effects of possible errors in estimating the critical volume and regimen of runoff. The rate of spillway discharge corresponding to any maximum reservoir level may be read from Fig. 19*b*, as indicated by the broken line in Fig. 19.

For use in estimating the duration of high reservoir stages and rates of outflow during the spillway design-flood, hydrographs similar to those shown in Fig.

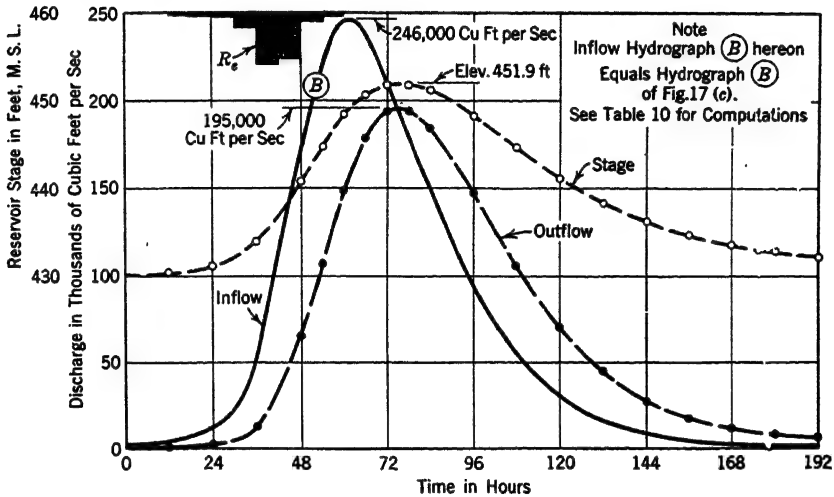


FIG. 20. Reservoir inflow, stage, outflow hydrographs.

20 are desirable. The time relation of the inflow, outflow, and stage hydrographs to the period of excessive rainfall during the design storm is also of importance in connection with the determination of operating schedules and flood-prediction services.

58. Freeboard. The term "freeboard" is defined as the difference in elevation between the top of the dam and the maximum reservoir level that would be attained during the spillway design flood. The freeboard should be adequate to prevent serious damage to the crest of the dam as the result of wave action and wind set up. These features are covered in Arts. 9 and 10 of Chapter 7.

59. Margin of Safety—General. In many cases, the failure of a high dam as the result of an inadequate spillway capacity would result in a major disaster, possibly involving a large loss of life. The probability of such a failure might be extremely low if the spillway were capable of passing the largest flood of record in the region, but in order to assure complete safety, the structure must be adequate to provide for the most critical conditions that can be anticipated, plus a reasonable margin of safety.

It is commonly acknowledged that parties responsible for placing structures in a natural stream channel are obligated to make certain that either hazards to downstream interests are not appreciably increased thereby or satisfactory compensations could be made for such damages as might result from operation or failure of the structures. In estimating spillway requirements for relatively low dams, the failure of which would not cause extreme property damage or constitute a serious threat to life, the margin of safety may be made consistent with economic analyses. However, where high embankments are involved, the damages that would result from failure of the dam because of an inadequate spillway would be greater than could be repaired by the owners of the project, even though equitable compensations were practicable in such cases. The social repercussions and hazards to life resulting from such a failure are not susceptible of economic evaluation.

If danger to the structures alone were involved, the sponsors of many projects would prefer to rely on the improbability of an extreme flood occurrence rather than to incur the expense necessary to assure complete protection. However, when a major portion of the risks involve downstream interests, a very conservative attitude is required in developing spillway design-flood criteria. Probable future development in the downstream flood plain, as well as existing conditions, must be taken into consideration in estimating potential damages and hazards to human life that would result from failure of a dam.

In judging the adequacy of the spillway capacity and height of dam proposed for a particular project, it is necessary to take several items into account, such as the relation of surcharge storage to spillway discharge capacity, freeboard allowances, the ability of the crest of the dam to withstand wave action, and the probable accuracy of the adopted spillway design-flood estimate.

60. Relation of Surcharge Storage to Spillway Discharge. In the following discussion, the term "surcharge" refers to the height of the reservoir level above the normal full-pool elevation. The term "surcharge storage" relates to the volume of reservoir storage corresponding to the surcharge height. The full-reservoir stage usually corresponds to the sill-elevation of an uncontrolled spillway and to the top of crest gates on a controlled spillway, although there are exceptions to these rules.

The relative importance of an accurate estimate of critical *flood volume*, as compared with an estimate of critical runoff *rates* for a particular drainage basin, is determined by the relation of the surcharge storage capacity of the reservoir to the spillway discharge capacity at corresponding elevations. If the height of a particular dam were sufficient to impound the entire volume of the maximum probable flood with zero outflow, the maximum reservoir elevation attained would depend upon the *volume* of the flood and in no respect would be influenced by the rate of inflow. In such a case, the accuracy of an estimate of the highest reservoir level during the maximum probable flood would depend entirely upon the accuracy with which the volume of the flood runoff, the initial pool level, and the storage-capacity curve could be determined. On the other hand, if a gate-controlled spillway were provided and operated in such

a manner as to prevent the accumulation of surcharge storage, the maximum rate of outflow would equal the maximum rate of inflow, regardless of the volume of the flood. In this case, the reliability of estimates of spillway requirements would be dependent upon the accuracy of the spillway rating curve and the accuracy of the estimated peak rate of runoff. In most reservoir projects, the relation of surcharge storage to spillway discharge capacity falls between the two extremes referred to above.

On the basis of the discussion given in the preceding paragraph, the following generalization may be stated: If the surcharge capacity of a reservoir is relatively large in proportion to the spillway discharge during the critical period of major floods, an accurate estimate of the *volume* of the spillway design flood is more important than an accurate determination of the rate of inflow; and, conversely, if the spillway discharge is relatively high in proportion to the volume of surcharge storage, an accurate determination of the critical *rate* of runoff is of greatest importance. (See Art. 57.)

61. Freeboard Allowances. Freeboard provided for wave action should not be infringed upon during the spillway design flood. However, under some circumstances freeboard allowances may be considered as constituting a margin of safety. In such cases it should be shown that the occurrence of wind velocities capable of producing maximum waves at the time when the reservoir level is near the maximum is very improbable.

The security of a dam against damage by wave action is not determined entirely by the height of freeboard provided. The ability of the dam to withstand erosive action of waves is of equal or greater importance. Protective measures should be ample to prevent dangerous erosion by both direct action of the waves on the face of the dam and the wave-splash that may be carried over the crest of the dam during critical periods.

62. Accuracy of Spillway Design-Flood Estimates. Matters determining the reliability of spillway design-flood estimates are summarized as follows:

(*a*) *Scope and accuracy of hydrologic records used in the basic studies.* It is reasonable to assume that maximum probable flood estimates based on a long period of hydrologic records are more reliable than those founded on shorter records. The availability of reliable basic records for a representative range of conditions makes possible hydrological studies that may eliminate many uncertainties regarding runoff characteristics of the drainage basin involved. However, a long period of record does not necessarily reflect critical flood potentialities of a region. For example, three of the greatest floods observed within approximately the last 300 years in the northeastern United States occurred during the years 1936 and 1938.

(*b*) *Thoroughness of basic meteorological and hydrological studies.* To aid in avoiding gross misconceptions of flood potentialities of a basin, records of floods should be supplemented by meteorological analyses. The combinations of circumstances resulting in maximum floods are unusual and comparatively infrequent. The infrequencies of such phenomena make it very difficult, or entirely impracticable, to obtain a reliable estimate of the maximum probable

flood in a particular basin on the basis of hydrologic records available for that area alone. The investigation of major storms and floods of record in a general region surrounding the basin under study serves to supplement data available for the specific area, and may increase the reliability of the results.

(c) *Experience and judgment of personnel engaged in the interpretation and analysis of data.* Knowledge gained by training and experience in hydrological and meteorological studies adds greatly to the reliability of design-flood estimates. Because of the diverse character of the many hydrological and meteorological phenomena, it is seldom that any one individual can gain comprehensive experience in the solution of problems pertaining thereto. It is therefore of more than usual importance that one's own experience should be supplemented by a thorough study of technical publications containing accounts of the findings of other investigators.

(d) *Size of basin, physical characteristics affecting infiltration and runoff therein, and the meteorological conditions characteristic of the region.* It is more difficult to obtain reliable estimates of critical-flood magnitudes in some basins than in others. For example, there is a relatively small range for error in estimating minimum infiltration losses for basins in which it is known that 80 to 90 per cent of storm rainfall normally runs off, whereas the possible error is very large in estimating losses for basins characterized by high infiltration capacities. As a general rule, the *percentage* of error in estimating peak rates of runoff from small basins is likely to be greater than for large basins. This is true for several reasons: the difference between normal and extreme rainfall rates is greatest over small areas, the modulating effect of valley storage is usually less in small basins, and the knowledge of major floods on small drainage areas is less complete because of the extreme difficulty of obtaining accurate measurements of sudden flood rises characteristic of such areas. In basins having large valley storage capacities, major variations in rainfall intensity may have a small influence on discharge rates, whereas the same variations in other basins might result in appreciable differences.

63. Margin of Safety—Summary. It is evident from the preceding discussion that rigid rules cannot be prescribed for fixing the proper margin of safety to be represented in a spillway design-flood estimate. The final decisions must, to a great extent, be based on judgment. Inasmuch as the purpose of a spillway design-flood estimate is to determine the spillway capacity and height of dam required to provide safely for extreme eventualities, the collective influence of the factors involved is of ultimate concern, rather than the individual effects. However, in order to judge the degree of conservatism inherent in a spillway design-flood estimate, it is necessary to evaluate the effect of reasonable variations in the principal factors involved. The computation procedures outlined in the preceding paragraphs were developed to permit a careful examination of the individual conditions and combinations of events that result in critical-flood hydrographs in a particular drainage basin.

64. Selection of Spillway Design-Flood Hydrograph. The "provisional spillway design-flood" hydrograph is derived to represent the critical volume and

concentration of runoff from the most severe combination of meteorological and hydrological conditions that can be anticipated on the basis of available data. Inasmuch as computation of the provisional spillway design-flood hydrograph involves consideration of hydrological and meteorological influences only, it may be assumed that the hydrograph would be the same regardless of the type of dam, size and type of spillway, and height of freeboard adopted for a particular project. However, the final spillway design-flood hydrograph is intended to reflect all margins of safety that are considered necessary in a particular instance to assure the desired degree of protection against severe damage to or failure of the dam.

Inasmuch as the extent of modifications in the provisional spillway design-flood hydrograph necessary to reflect the desired margins of safety are dependent to a great extent upon the features of a particular project, the selection of the final spillway design-flood criteria should not be based entirely upon hydrological and meteorological studies. Different spillway design-flood criteria may be advisable for alternate designs of projects for the same site, if the safety features of alternate designs differ greatly. However, in view of the many uncertainties involved in such studies, it is generally customary to select a spillway design-flood hydrograph that is considered sufficiently conservative for use in determining the most economical combination of spillway capacity and height of dam, without revisions in the hydrograph to obtain entirely comparable margins of safety for relatively minor differences in design assumptions.

In connection with the sample computations discussed in Art. 51, hydrograph B of Fig. 17*c* was selected as the spillway design-flood hydrograph for the project involved. The factors discussed in Arts. 59 to 63 were taken into consideration in selecting hydrograph B. Minor revisions in the tentatively adopted spillway capacity and height of dam were made to obtain the most economical design that would provide safely for the design flood.

III. BIBLIOGRAPHY

65. General. The following publications are referred to in the text of this chapter by numbers. Other references are given throughout the text.

1. ROBERT E. HORTON, "Determination of Infiltration Capacity for Large Drainage Basins," *Trans. Am. Geophys. Union*, Part 2, 1937, pp. 371–385.

2. ROBERT E. HORTON, "Analysis of Runoff-Plot Experiments and Varying Infiltration Capacity," *Trans. Am. Geophys. Union*, Part 4, 1939, pp. 693–711.

3. L. K. SHERMAN and L. C. MAYER, "Application of the Infiltration Theory to Engineering Practice," *Trans. Am. Geophys. Union*, Part 3, 1941, pp. 666–677.

4. L. K. SHERMAN, "The Unit Hydrograph and Its Application," *Bulletin of Associated State Engineering Societies*, April 1941.

5. G. R. FREE, G. M. BROWNING, and G. W. MUSGRAVE, "Relative Infiltration and Related Physical Characteristics of Certain Soils," *U. S. Dept. Agr. Tech. Bull.* 729, July 1940, Washington, D. C.

6. FRANKLIN F. SNYDER, "A Conception of Runoff Phenomena," *Trans. Am. Geophys. Union*, Part 4 1939, pp. 725–738.

7. FRANKLIN F SNYDER, "Synthetic Unit Graphs," *Trans. Am. Geophys. Union*, Part 1, 1938, pp. 447–454.

8. E. J. RUTTER, Q. B. GRAVES, and F. F. SNYDER, "Flood Routing," *Trans. Am. Soc. Civil Engrs.*, Vol. 104, 1939, pp. 275–313.

9. W. B. LANGBEIN, "Some Channel Storage Studies and Their Application to the Determination of Infiltration," *Trans. Am. Geophys. Union*, Part 1, 1938, pp. 435–447.

10. B. R. GILCREST and L. E. MARSH, "Channel-Storage and Discharge-Relations in the Lower Ohio River Valley," *Trans. Am. Geophys. Union*, 22nd Ann. Meeting, 1941.

11. BERTRAM E. BARNES, "The Structure of Discharge-Recession Curves," *Trans. Am. Geophys. Union*, Part 4, 1939, pp. 721–725.

12. JEROME NAMIAS, *An Introduction to the Study of Air Mass and Isentropic Analysis*, 5th Edition (Oct. 1940), published by the American Meteorological Society, Milton, Mass.

13. *A Report on the Maximum Possible Precipitation over the Ohio River Tributary Basins above Pittsburgh, Pa.*, published report prepared by the Hydrometeorological Section of the U. S. Weather Bureau in cooperation with the U. S. Engineer Department.

14. *A Report on the Maximum Possible Precipitation over the Ompompanoosuc Basin above Union Village, Vt.*, published report prepared by the Hydrometeorological Section of the U. S. Weather Bureau in cooperation with the U. S. Engineer Department.

15. "Storm Rainfall of Eastern United States," *Miami Conservancy District, State of Ohio, Tech. Repts*, Part 5 (revised, 1936), by the Engineering Staff.

16. S. M. BAILEY and G. R. SCHNEIDER, "The Maximum Probable Flood and its Relation to Spillway Capacity," *Civil Eng.*, January 1939, p. 32.

17. GAIL A. HATHAWAY, "Applications of Hydrology in Flood Control," *Proc. Hydrology Conf.*, June 30–July 2, 1941, held under auspices of Pennsylvania State College, Am. Soc. Civil Engrs., Am. Geophys. Union, and Soc. for Promotion of Eng. Educ.

18. MERRILL BERNARD and WALTER T. WILSON, "A New Technique for the Determination of Heat Necessary to Melt Snow," *Trans. Am. Geophys. Union*, Part 1, 1941, pp. 178–181.

19. W. T. WILSON, "An Outline of the Thermodynamics of Snow-Melt," *Trans. Am. Geophys. Union*, Part 1, 1941, pp. 182–195.

20. PHILLIP LIGHT, "Analysis of High Rates of Snow Melting," *Trans. Am. Geophys. Union*, Part 1, 1941, pp. 195–205.

21. ROBERT E. HORTON, "Phenomena of the Contact Zone Between the Ground Surface and a Layer of Melting Snow," *Intern. Assoc. Hydrol. Bull.* 23, 1938, pp. 545–561.

22. U. S. Geol. Survey Water-Supply Papers 798, 799, and 800.

23. GEORGE D. CLYDE, "Snow-Melting Characteristics," *Tech. Bull.* 231, August 1931, *Utah Agr. Exp. Sta.*, Utah State Agr. College, Logan, Utah.

24. OTTO N. RUGEN, "The Snow-Melt Problem as Affecting the Design of Flood Control-Works" (Discussions by J. E. CHURCH and H. U. SVERDRUP), *Trans. Am. Geophys. Union*, Part 3, 1940, pp. 1033–1046.

25. *Report on Partial Survey of Tennessee River and Tributaries*, H.D. 185, 70th Cong., 1st Sess., Appen. B, Part 2, pp. 46–62.

26. C. S. JARVIS, "Floods in the United States," *U. S. Geol. Survey Water-Supply Paper* 771, 1936.

27. W. G. HOYT, "Studies of Relation of Rainfall and Run-Off in the United States," *U. S. Geol. Survey Water-Supply Paper* 772, 1936.

28. MERRILL BERNARD, "Formulas for Rainfall Intensities of Long Duration," *Trans. Am. Soc. Civil Engrs.*, 1932, p. 592.

29. GREGORY and ARNOLD, "Run-Off—Rational Run-Off Formulas," *Trans. Am. Soc. Civil Engrs.*, 1932, p. 1038.

30. D. L. YARNELL, "Rainfall Intensity-Frequency Data," *U. S. Dept. Agr., Misc. Pub.* 204, 1935.

31. C. S. JARVIS, "Flood Flow Characteristics," *Trans. Am. Soc. Civil Engrs.*, 1926 p. 985.

32. ALLEN HAZEN, *Flood Flows*, John Wiley & Sons, 1930.

33. CARL G. PAULSEN, "The Measurement and Computation of Flood-Discharge," *Trans. Am. Geophys. Union*, Part 2, 1939, p. 177.

34. "Report of Committee on Floods," *J. Boston Soc. Civil Engrs.*, Vol. 17, No. 7, September 1930.

35. *Bull. Am. Meteorol. Soc.*, Vol. 17, Nos. 6 and 7, June, July 1936.

36. MERRILL BERNARD, "An Approach to Determinate Stream Flow," *Trans. Am. Soc. Civil Engrs.*, 1935, p. 347.

37. "Floods in the United States" (detailed descriptions), *U. S. Geol. Survey Water-Supply Papers* 88, 92, 96, 147, 162, 334, 487, 488, 771, 798, 799, 800, and 816.

38. WILLIAM P. CREAGER, "Possible and Probable Future Floods," *Civil Eng.*, November 1939, p. 668.

39. CREAGER and JUSTIN, *Hydro-Electric Handbook*, John Wiley & Sons, 1927.

40. Appendix 7, Flood Commission, Pittsburgh, Pa.

41. I. E. HOUK, "High Rainfalls and Runoff from Small Areas," *Eng. News-Record*, Vol. 87, 1922, p. 480.

42. E. F. MCCARTHY, "Forest Cover and Flood Control," *Trans. Am. Soc. Civil Engrs.*, 1929.

43. H. P. EDDY, JR., "New England Floods of 1936," *Civil Eng.*, May 1936.

44. "National Aspects of Flood Control, a Symposium," *Trans. Am. Soc. Civil Engrs.* 1938.

45. W. E. FULLER, "Flood Flows," *Trans. Am. Soc. Civil Engrs.*, 1914, p. 564.

CHAPTER 6

SPILLWAYS *

1. Chute Spillways. For dams composed of earth or rock, or for dams over which it is impossible or undesirable, for special reasons, to pass floods, some form of spillway adjacent to the dam must be adopted. One general type consists of a low spillway controlling an inclined slope on the natural or excavated earth or rock formation to convey the water to the river below the dam. Where the site for such a spillway is limited in width, the side channel spillway, described in Art. 2, is adopted; but, where ample room is available the chute spillway is used. These spillways are located where the contours, both earth and rock, are best suited for economical construction.

Fig. 1*a* shows the layout of the chute spillway for the Tionesta, Pa., Dam. It is located where the divide is low and where the discharge at its lower end is remote from the toe of the dam.

The term "chute spillway," although generally adopted, is a misnomer since many spillways, such as the side-channel spillway and even a narrow spillway over a concrete dam, have the "chute" or trough characteristics on the discharge end. However, the term as here used refers to a spillway isolated from the dam, having its crest normal to its center line, as in Fig. 1*a* and having a discharge channel to the river in an excavated trench which is usually paved with concrete in whole or in part. The crest or spillway proper is usually of insignificant height or actually flat, as in Fig. 1*b*.

A thorough study of the chute spillways of this country was made by A. L. Alin, U. S. Engineer Office, Denison, Tex., in connection with the design of the Denison Dam spillway. His report, entitled *Report on Chute Spillways*, was published by that office in December 1939. It contains more data on existing spillways of this type than has ever been accumulated before. It has been drawn upon freely for data for use in this article.

(*a*) *Foundations.* A chute spillway may be constructed on any type of foundation capable of sustaining the load without undue deformation. However, if it is incapable of passing the greatest flood without excessive erosion, it must be protected by a concrete paving.

Few data are available on the relative erodibility of earth and rock. Uncemented sands and gravels are, of course, the most easily eroded. Hardpan and clays are much more resistant to flowing water. Undisturbed fat clays and shales have been found on the bottoms of many swiftly flowing streams. Horizontally stratified hard rock, with many vertical seams to permit the entrance of water under high velocity, is the most easily eroded of the hard

* Special types of spillways only. See also Chapters 11, 14, and 24.

rocks. The velocity of the water entering the cracks creates dynamic pressure which is transferred to the horizontal strata and may lift such rock out in large

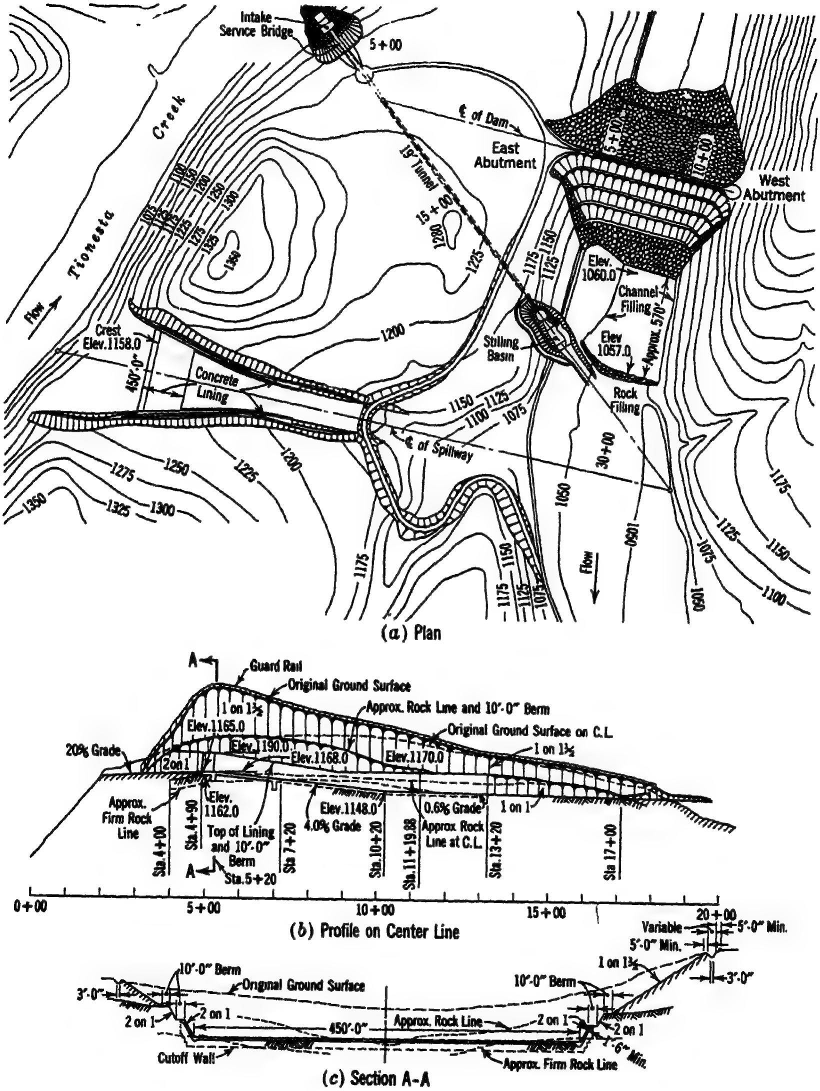


FIG. 1. Tionesta chute spillway (U. S. War Dept.).

blocks. Vertically stratified rock and, of course, solid rock are the most resistant.

In general, the degree of protection against erosion is governed by probable cost of maintenance rather than complete failure. Hence the frequency of

operation of the spillway becomes an influencing condition. For instance, in some flood-control dams, the estimated average frequency of any use of the spillway is once in several hundred years, whereas for dams primarily for power, small floods every year and large floods every few years are expected.

Certain clays and shales are subject to swelling when the weight of the excavation for the discharge trough is removed and when they become saturated. In such places special attention must be paid to detail of the concrete paving, and particularly where it joins the side slope paving or side retaining walls, to guard against cracking due to such movement.

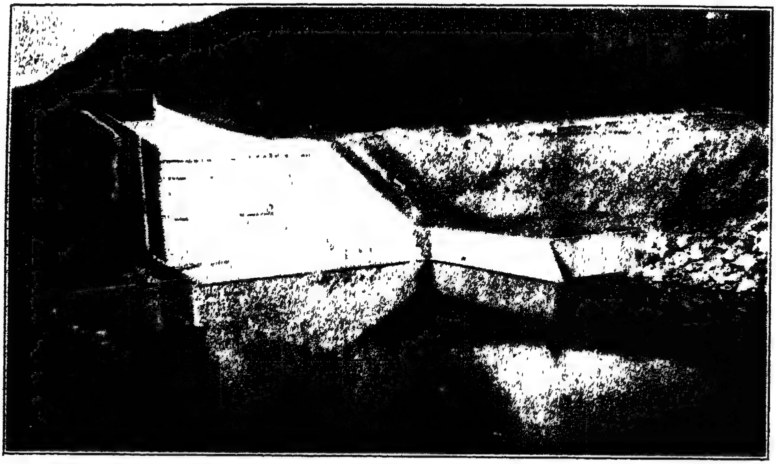


FIG. 2. Chute spillway for Kochess Dam (*A. L. Alin, loc. cit.*).

The protective paving is sometimes carried throughout the entire length of the spillway and, in addition, stilling basins (Art. 37, Chapter 3) have been provided at the lower end of the spillway to dissipate the energy of the water and to avoid undermining the paving at that place. Such a spillway is shown in Fig. 2. In other places, as shown in Figs. 1 and 3, the paving has been provided for only a short distance below the crest, any damage below the paving being considered not a menace to safety of the spillway and not a considerable maintenance factor.

(*b*) ***Failures.*** Of the 83 chute spillways studied by Alin, 12 had failed or had experienced trouble. Of these, 3 were built on embankments, a procedure which is not considered good practice, 3 scoured the foundation badly downstream from the paving, 1 was ruptured by uplift from headwater owing to inadequate upper cutoff and drainage, 1 failed because of high velocity of water entering cracks in the paving, 1 had its side walls overtopped at a bend in the spillway, and the other 3 failed from indeterminate causes, but probably owing to cracking of the apron.

(*c*) ***General arrangement.*** The spillway crest may be straight, as in Fig. 1, but in some places an arc normal to the flaring side walls may be found more

economical. Where the crest is not a flat-crested weir but is a weir section of some height, the curved crest offers better hydraulic features, since it prevents disturbances downstream owing to the discharge over the crest not being parallel to the side walls.

The required length of crest is subject to economic considerations since, for a given elevation of crest, the height of the dam increases as the crest is shortened. The spillway is usually widest at the crest and then narrows to a width which is determined by the most economical shape of the discharge trough. A flare or widening at the extreme lower end is sometimes provided to reduce the energy per linear foot of the water entering a stilling basin.

FIG. 3. Chute spillway for San Gabriel No. 1 Dam (*A. L. Alin, loc. cit.*).

The center line of the spillway is usually straight, as in Fig. 1. When it must be curved, owing to unavoidable circumstances, particular attention must be paid to the degree of superelevation of the water surface at the outside of the bend. Hydraulic model tests are necessary to determine this closely. Sloped sides of the spillway trough should be avoided on the outside of sharp bends, because they cause a higher superelevation of water surface than vertical walls.

The slope of the chute is usually made as gentle as is consistent with the proper conveyance of the water to a point as far away from the crest as possible and then as steep as is consistent with stable slopes to reach the lower level. This arrangement reduces the excavation to a minimum, transfers the highest velocities as far as possible from the crest, and increases the distance that scour would be required to work back to the reservoir. However, in special cases, the steep portion of the chute is placed directly downstream from the crest in order to reach bedrock or a layer of rock which is not susceptible to scour as soon as possible. This reduces the length of concrete paving, which is often more expensive than the excavation.

In the latter case, a pilot channel is all that is required downstream from the foot of the steep slope, provided the unexcavated portion of the chute will be sure to scour to adequate capacity before the peak of the flood is reached. However, attention must be directed to possible damage due to the deposition of this scoured material downstream from the dam.

(*d*) *Thickness of concrete paving.* Where the spillway is on easily eroded earth, the use of the paving is obvious. Where rock is encountered, the use of the paving is to provide a smooth surface for the spilling water but principally to prevent water at high velocities from entering cracks in the rock and lifting out large masses, as previously explained. Where the spillway is subjected to high velocities, the paving cannot possibly be heavy enough to resist being lifted out should high velocities cause pressures which penetrate through cracks to the underside and subject it to the corresponding uplift. However, this upward pressure may be balanced by providing hold down piles if the foundation is earth or anchorage rods if the foundation is rock as described in the paragraph under "Anchorage."

Thus the thickness and weight of the paving is of less importance than its ability to withstand unequal settlement and cracking and still remain watertight. Of course, it must be thick enough to withstand weathering. Also it must be heavy enough to resist ground-water pressure; but with an adequate drainage system this is negligible; and, where some question may exist, anchor rods are used as subsequently explained.

There is no rational method for determining the thickness of concrete paving for chute spillways to conform to the specified local conditions. In fact, there is no established precedent, the thickness adopted for various similar spillways varying greatly, according to the ideas of the designers. Even the use of anchors to tie the paving to the foundation, mentioned later, does not seem to have caused any consistent variation in adopted thicknesses.

Paving has ranged in thickness from 4 in. to 5 ft., but thicknesses of 4 in. are not recommended for use in cold or temperate climates under any conditions. The variation in thickness is due in part to variation in foundation conditions and in part to the differing ideas of designers. Slope paving may be thinner at the top of the slope than at the bottom. Thickness of thoroughly reinforced concrete paving properly anchored to rock may logically be thinner than when the foundation is poorly consolidated earth of unequal bearing power.

(*e*) *Reinforcement for paving.* Where the pavement is divided into panels, separated from each other by contraction joints as described in sections *f* and *g* following, there is no mathematical basis for computing the amount of reinforcing steel. About one-fourth of 1 per cent each way, located in the top of the slab, is the general practice. The amount should be sufficient to avoid separation should an intermediate crack occur and to afford strength to bridge over any area of accidental local settlement.

Some designers under certain circumstances prefer to omit contraction joints and to reinforce the paving continuously from end to end, or over relatively long distances. In such case the criterion fixing the amount of steel to be used is that the total tensile strength of the steel, at its elastic limit, must

exceed the total tensile strength of the concrete at its breaking stress. A difficulty with this type of construction is that the tensile strength of the concrete is variable and unknown. The general practice is to use 0.5 or 0.6 of 1 per cent of *high elastic* limit (50,000 psi or more) steel for concrete having an ultimate *compressive* strength of 2,500 psi. This corresponds to a tensile strength for the concrete equal to about 0.1 the compressive strength. In this type of construction, it is important to limit the maximum as well as the minimum strength of the concrete.

(*f*) *Size of panels.* To avoid cracking, the concrete paving should be poured in square panels with contraction joints on all sides. Dimensions of 20 to 50 ft have been adopted variously with about 30 ft as a fair average. With careful curing of the concrete, and with reinforcement as mentioned in section *e*, 40 ft should be amply safe.

(*g*) *Joints.* Modern practice leans toward the use of contraction joints on all sides of the panels, with no through reinforcement and with the surface of the joint treated to permit free movement due to contraction and expansion. The type of joint to be adopted varies with the conditions at the site. For transverse joints (normal to the direction of flow) it is very essential that heaving or settlement of the foundation shall not cause the upstream surface of any panel to project above the surface of the panel next upstream, since this will result in a vertical face upon which the water at high velocity can impinge, resulting in pressure head in the joint which may be transmitted to the underside of the paving and cause uplift.

If the opposite is true (see Fig. 4), i.e., if the surface of the downstream panel at the joint is lower than that of the upstream panel, a suction or negative pressure will tend to occur in the joint which, if transmitted to the underside of the paving, would increase its stability. This step effect, as shown in Figs. 4 and 5, is now standard practice for transverse joints since it provides for slight differences in settlement or heaving and for errors in construction.

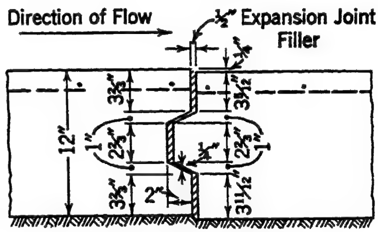


Fig. 4. Type A contraction joint.

On good rock foundations, where the chance of heaving or settlement is nil, transverse joints of the type shown in Fig. 4 may be used. However, if there is any uncertainty with regard to the foundation, or if a cutoff as explained subsequently is required, the joint shown in Fig. 5 is preferred. This type insures that the surface of any panel will not rise above the surface of the panel next upstream as long as the cutoff remains intact.

For longitudinal joints, which are parallel to the direction of flow, the item of unequal settlement, as affected by high velocity, is not so important, and the joint shown in Fig. 6 is adaptable.

Many other types of joints have been used according to the particular ideas of the designer. Dowels of round bars or structural steel have been used

across joints to prevent differential vertical movement. Such dowels, if used, must be treated throughout their imbedment in one of the panels to provide free movement of the joint due to expansion and contraction. It is felt that such dowels are generally unnecessary appurtenance if keyed joints are used.

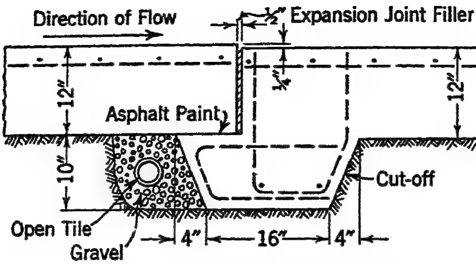


FIG. 5. Type B contraction joint.

Joint fillers capable of expansion and contraction must be provided in all joints, as shown in the illustrations. Many types are on the market. They must be elastic enough to stand compression without squeezing out of the joint and must respond to subsequent expansion. They must be strong enough not to be pulled out of the joint by the high velocity flow.

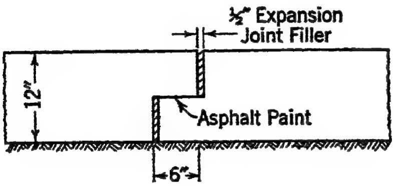


FIG. 6. Type C contraction joint.

Metal or rubber sealing strips, similar to those sometimes used in joints of dams (Art. 1, Chapter 23), have been adopted to make the joints watertight. However, this detail is of doubtful utility if the joint is otherwise well constructed.

(*h*) *Cutoffs*. Three types of cutoffs are necessary in chute spillways.

1. A cutoff at the upper end to prevent entrance of headwater pressure to the underside of the paving in amounts which would overstress the underdrains. This feature is no different from that employed in any gravity dam as described in Chapter 3.

2. In the absence of a stilling basin, a cutoff at the downstream end of the paving, consisting of a concrete-filled trench, designed to prevent undercutting of the paving. Such a cutoff is shown in Fig. 1*b*. The depth and thickness of such a cutoff vary with the nature of the foundation, there being no established precedent. The adopted depths have ranged from a few feet to as much as 70 ft. However, excessive depths of cutoffs may be avoided usually by extending the paving or providing a stilling basin.

3. Expansion and contraction due to changes in temperature tend to make the concrete paving panels creep if built on slopes. Therefore, under such conditions, a cutoff at the upstream end of each panel is necessary to prevent such movement as well as to prevent flow from one panel to another

along the underside of the paving. They are of course an absolute necessity where the paving rests on a continuous layer of gravel as subsequently mentioned. A typical cutoff of this type is shown in Fig. 5. It is cast integrally with the downstream panel and strengthened by ample reinforcing steel.

(*i*) *Drainage.* A drainage system is necessary under the paving to prevent uplift from ground water or water that finds its way through the paving during operation of the spillway. If the paving is on a rock foundation, the drainage system generally consists of gravel-filled trenches under the paving, with sometimes an open tile drain imbedded in the gravel. The transverse drains of this type are usually placed along the upstream side of each cutoff, as shown in Fig. 5. Similar longitudinal drains are also provided, forming a network under the paving. It is sometimes advisable to locate drains at places where a crack in the rock indicates possible future seepage.

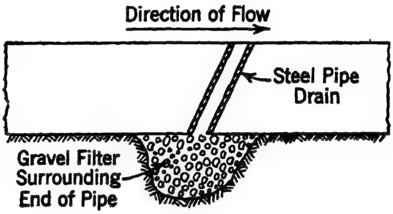


FIG. 7.

The drains are either relieved at intervals through the paving, as indicated in Fig. 7, or are collected into one or more trunk drains which carry the entire flow to an outlet at the lower end of the chute. The latter method is preferred by some on the ground that the small drains through the paving may become plugged by animals, sediment, or by other obstructions. If the former method embodying individual outlets is used, the drainage system for each paving panel should be independent in every way of that of a panel at a lower level, so that water entering the system at an upper level will not cause pressure at lower levels. All outlet pipes, of the type shown in Fig. 7, should be given a downstream inclination so that the high velocity of the water flowing in the chute will tend to cause a suction in them.

With the trunk drainage system, the pipes should be designed to flow only partly full for the greatest possible discharge entering at all places, in order to insure against uplift.

If the foundation consists of poor rock or pervious earth, a 6- to 12-in. layer of gravel is sometimes placed under the entire paving, properly relieved at intervals by tile pipe or pipes through the paving. Care should be taken to prevent the gravel from being washed or sucked into the pipes by installing a graded gravel filter or screen at the entrance to the pipes. Tar paper may be placed over the gravel to prevent the mortar from penetrating into the gravel when the apron is placed.

Where the rock is horizontally stratified and ground water may cause uplift on layers of rock below the apron and out of contact with the drains, drainage holes have been drilled in the bottoms of the drainage trenches to relieve the underpressure.

(*j*) *Anchorages.* Anchorages for the concrete paving consist of reinforcing steel grouted into holes drilled in the rock and tied to the concrete. Tests

should be made to determine the depth to which they should be imbedded to resist their ultimate strength. About 50 per cent of chute spillway paving on rock foundations has been anchored. Where such anchors have been used, the area of paving controlled by 1 sq in. of anchor has ranged from 20 to nearly 200 sq ft but has averaged about 80 sq ft. Such anchors are generally desirable where the amount of ground water to be controlled by the drains might be sufficient to result in ground-water uplift on the paving.

(*k*) *Slopes of the cut.* Obviously the slopes adopted for and above the structure of chute spillways must be very gentle. There is no structure that is more dependent on this feature, since any slide that would damage the paving during a flood would spell disaster.

(*l*) *Hydraulics of chute spillways.* Theories of discharge over standard-crest spillways are covered in Art. 3 of Chapter 11. Where a flat-crested spillway is used, its discharge is

$$Q = 3.087LH^{3/2}$$

where Q = discharge in sec ft;
L = spillway length, in feet, corrected for end contractions, if any;
H = head, in feet on the crest, measured to the elevation of water surface in the reservoir, but reduced by the amount of friction loss between the reservoir and the crest.

The depth of water at the downstream toe of a standard spillway is covered in Art. 28 of Chapter 3. The depth below the flat-crested spillway is equal to $\frac{2H}{3}$. The flow over the standard spillway will be considerably below the critical depth; but over the flat-crested spillway it will be just at the critical depth. In either spillway, the slope of the chute below the crest should be sufficient to carry away the discharge at slightly less than critical depth in order to prevent undulating flow. Further hydraulic calculations are no different from those required for flumes flowing at less than critical depth. However, one should be sure that vertical curves in the chute are within the trajectory of the flow, where the velocities are high, so that the underside of the flowing sheet will not leave the paving and cause a vacuum.

(*m*) *Hydraulic model tests.* With proper care in hydraulic studies by a competent engineer, and particularly where the shape of the crest and other features are such that precedent is available for the adoption of hydraulic coefficients, hydraulic model tests are not necessary. However, for unusual conditions and wherever there is a bend in the chute, appropriate model tests should be made.

2. Side-Channel Spillways. (*a*) *Description of type.* The term "side-channel spillway" is used to designate structures of the general type shown in Fig. 8. Its distinguishing characteristic is that the flow, after passing over a weir or ogee crest, is carried away by a channel running essentially parallel to the crest. The type is suitable for earth or rock-fill dams in narrow canyons, and for other situations where direct overflow is not permissible and where the

space required for a chute spillway (see Art. 1 of this chapter) of adequate crest length is not available.

Fig. 8 is based on the spillway for the Tieton Dam, an earth-fill structure in the state of Washington.[1]

(b) *Hydraulic theory.* At the moment that any portion of the flow over the crest reaches the main body of water already in the channel, it has an appreciable downward and transverse velocity. This velocity is of no assistance in

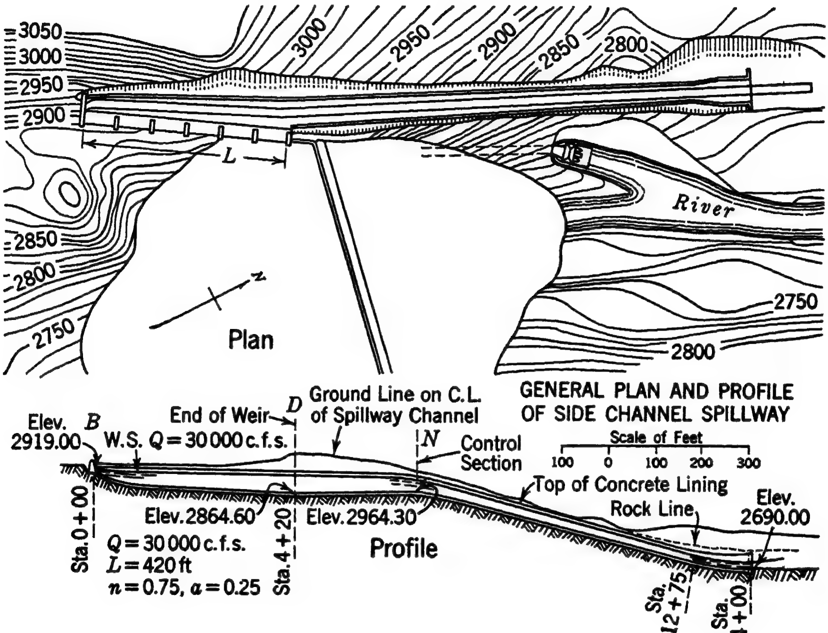


FIG. 8.

moving the water along the channel. Axial velocity is produced after the incoming particles join the channel stream. The accelerating force is derived from the surface slope.

Fig. 9 represents diagrammatically a section along the center line of a side-channel spillway. The water surface falls along some curve from the point B at the upper end of the channel to a point D opposite the downstream end of the crest. Each particle coming into the channel contributes to the production of velocity and the overcoming of resistances between B and D an amount of energy represented by its effective fall, or the drop in water surface from the point at which the particle enters to D. Thus a particle coming in at B will have an effective fall equal to PD, whereas one coming in at D will have no effective fall. The total applied energy down to the point D is equal to that

[1] JULIAN HINDS, "Side Channel Spillways," *Trans. Am. Soc. Civil Engrs.*, Vol. 89, 1926, p. 881. (Figs. 9 to 14 are taken from this same paper.)

produced by the entire flow falling through the average drop for all the particles. If the surface curve from B to D is a straight line, and if the inflow per foot of crest is uniform, the average fall will be one-half the total fall PD. If the curve is convexed upward, as BCD, the average fall will be greater than one-half PD, and if the curve is concaved, the proportion will be less than one-half PD.

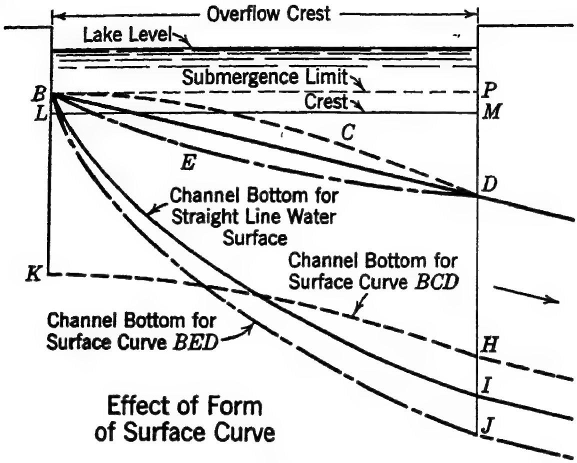


FIG. 9.

(c) *Hydraulic formulas.*

(1) *General.* Only a part of the average fall is available for the production of velocity head at D. The remainder is used to overcome resistances. A resistance due to equalization of relative axial velocities between incoming water and water already in the channel exists at every point along a side-channel spillway from the beginning to the end of the crest. This resistance will be referred to as "inertial resistance." Flow in such case is determined by making the momentum at the end of any short reach equal to that at the beginning of the reach plus any increase due to external forces. "External forces" include inertial forces, gravity, and friction. Frictional resistances are relatively small and may be neglected or estimated by methods used for other variable flows.

For a frictionless channel, it can be shown [2] that the theoretical ordinate y of the water surface curve, measured downward from the line B–P, Fig. 9, is given by the equation

$$y = \frac{1}{g}\int_0^x \left(V\frac{\partial V}{\partial x} + \frac{q}{Q}V^2 \right) \partial x \qquad [1]$$

where V = *average* velocity of water particles in direction of channel,
q = inflow in a unit distance, and
Q = total flow at point corresponding to y.

[2] JULIAN HINDS, op. cit.

In momentum problems, the "average velocity" is the arithmetical or algebraic average of the velocities of all the particles, in the direction under consideration. Unless velocities are uniform throughout the cross-section of the stream (a condition which never occurs), the *average* velocity is not the same as the *mean* velocity, which is obtained by dividing discharge by area. In a side-channel spillway, velocity distribution is unusually irregular. However, the mean velocity is usually used because of ease of computation. The resulting accuracy may be considered consistent with the general degree of accuracy in flood-flow problems.

If the relations of Q and V to x are known in a given case, Eq. 1 can be integrated and the form of the surface curve determined. These relations depend on the form and dimensions of the channel, and are algebraically complicated unless the channel is purposely designed to make them simple.

(2) *Simple new design.* In preparing an unrestricted new design, computation may be facilitated by arbitrarily establishing a simple relationship between Q, V, and x and proportioning the structure to correspond.

The inflow per foot of spillway crest for the purpose of design will generally be uniform, and the total discharge at a section x distant from the upper end of the crest will be given by an equation of the form

$$Q = qx \tag{2}$$

where Q is the total flow at the point and q is inflow per foot of crest.

An equation of the exponential type will be found convenient for expressing the velocity-distance relation, and by properly choosing constants, such an equation can be made to conform to a sufficient range of conditions to meet usual requirements. The following form is suggested:

$$V = ax^n \tag{3}$$

where a and n are arbitrary constants and V and x denote, respectively, average velocity and distance from the upper end of the crest.

Substituting these values for V and Q in Eq. 1, integrating, and reducing,

$$y = \frac{a^2(n+1)}{2gn} x^{2n} \tag{4}$$

Noting that $a^2x^{2n} = V^2 = 2gh_v$, Eq. 4 may be simplified to

$$y = \frac{n+1}{n} h_v \tag{5}$$

in which h_v is the theoretical velocity head.

(*d*) *Economic factors, new design.* A spillway channel is completely determined by Eqs. 2 to 5, if its cross-sectional shape and the values of a and n are chosen. The proper choice of these factors is controlled by economic considerations. The discussion will be confined to a trapezoidal channel on a compara-

tively steep hillside, which is a usual case. The conclusions reached can be revised to suit other conditions.

The effect of the shape of the channel on excavation quantities is illustrated in Figs. 10 and 11. Safety demands that the channel be set well into the

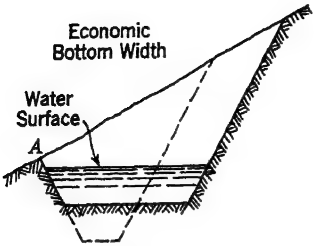


FIG. 10.

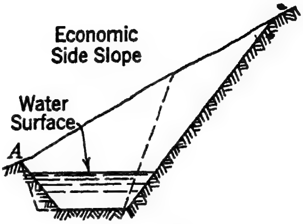


FIG. 11

original formation. If the water-surface elevation, channel side slopes, area of water prism, and location of point of outcrop are fixed, it is evident from Figs. 10 and 11 that the excavation is reduced by a narrow bottom width and steep side slopes. The minimum practical width of bottom depends on the excavating equipment. The side slopes should be trimmed to the steepest angle at which the materials will safely stand.

Usually it will be necessary to line the spillway channel with concrete. The cost of lining, which is an important item, is least when the bottom width is such that the wetted perimeter is a minimum.

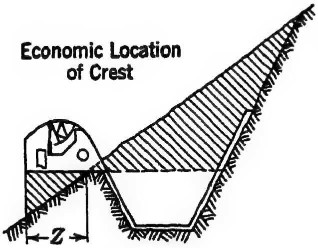


FIG. 12.

Figs. 10 and 11 represent that part of the channel downstream from the crest structure, but the same principles apply to the part of the channel opposite the crest.

With a movable crest of the drum type, an additional factor is introduced, owing to the necessity for supporting the crest on a concrete base. It will be found advantageous to set the channel back into the hill a certain distance to reduce the quantity of concrete required for this purpose. The distance Z in Fig. 12 is chosen by trial to make the combined cost of the shaded part of the concrete and the shaded part of the excavation a minimum.

(*e*) *Trial values of a and n.*

(1) *General relations.* After the cross-section of the channel has been selected, the profile is controlled by the values of a and n. Assuming a specified drop in the water surface from B to D, the effect of varying n is illustrated in Fig. 9. If $n = 0.5$, the surface curve is straight and the channel bottom will follow some such line as BI. If n is greater than 0.5, the water-surface curve will be convexed upward as BCD. If n is exactly 1.0, BCD will be a parabola and the bottom will be a parallel curve, as KH.

If n lies between 0.5 and 1.0, the bottom line will start at B. If n exceeds unity, the bottom line theoretically drops to an infinite depth at the upper end of the channel, then rises rapidly approximately to the line KH, which it crosses before reaching H. The effect on the profile of varying both a and n for the spillway of Fig. 8 is shown in Fig. 13.

The final selection of values of a and n for greatest economy depends on topography and can be made only by trial estimates. The value of n is less variable than a, hence it is usual first to assume a trial value for n, find the

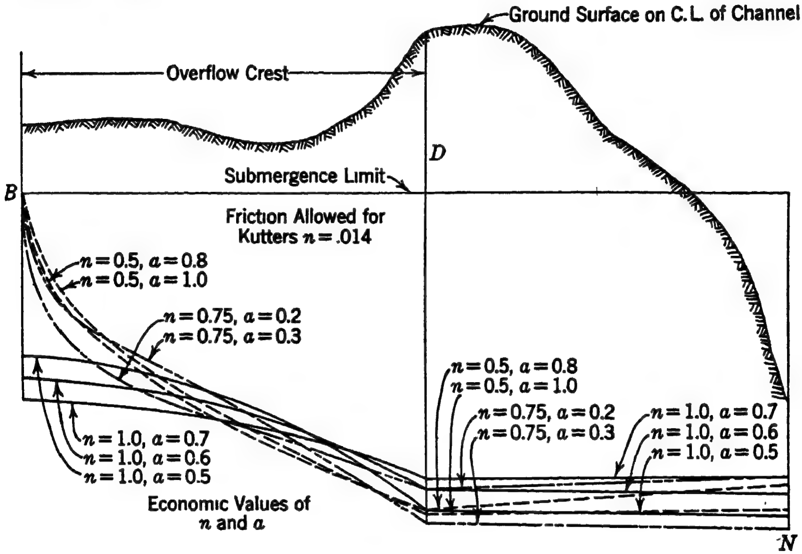


FIG. 13.

corresponding best value for a, and repeat until the best combination is found. The number of trials required can be reduced by a preliminary mathematical study.

(2) *Mathematical approximations.* It can be shown that $d + y$ will be minimum at any *specified single point* along the spillway crest when conditions are such that

$$h_v = \frac{n}{n+1}\frac{A}{2T} \qquad [6]$$

where A is the area and T is the top width of the water prism at the point; or, substituting in Eq. 5

$$y = \frac{A}{2T} \qquad [7]$$

Having chosen a value for either a or n, the corresponding value of the other required to give the greatest economy at a *specified point* can be found from these equations. Because of the algebraic complexity of general

equations for trapezoidal channels (the usual form), a trial procedure is best. First assume a depth of channel, compute A and T and find h_v (for a specified value of n), from Eq. 6; then compute the corresponding discharge from the equation

$$Q' = A\sqrt{2gh_v} \tag{8}$$

If Q' is equal to the required discharge, the trial depth is correct; otherwise, make a new trial. With V, x, and n determined, the corresponding value of a is computed from Eq. 3.

The values of a and n thus determined are based on computations at some selected single point. They are not necessarily the best values for the spillway as a whole, but if the point for which they are computed is properly chosen, the results will aid in the selection of trial values for the comparative estimates to be discussed in section *g*.

The channel usually must be set well back into the bank opposite the end of the dam, for safety; hence the excavation is likely to be deep at the downstream end of the crest, as at point D, Fig. 13. Consequently, this is a likely point for the application of preliminary Eqs. 6, 7, and 8, but other possible control points may require testing by trial estimate.

(*f*) *Hydraulic control point.* Generally, the depth at D, Fig. 13, determined as outlined, is greater than critical depth. Such a depth is not based on any hydraulic necessity but is chosen to give minimum excavation quantities, as previously explained. To this end it is evidently desirable to keep the floor level as high as possible throughout the reach of deep cutting, as from D to N, Fig. 8. This is accomplished by making the depth critical at N, and choosing a floor elevation at that point which will exactly maintain the required depth at D. A lower elevation at N would not interfere with hydraulic operation but would involve needless excavation.

The point of critical depth is called the hydraulic control. Its best location is usually at the point where grade must be steepened to keep the channel in the ground, i.e., at N, Fig. 8. However, special conditions may require another location. For example, in the Boulder Dam spillways (Fig. 15), the control is immediately downstream from the end of the crest.

Flow at the control point must be at critical depth, which occurs when

$$h_v = \frac{A}{2T} \tag{9}$$

where symbols are as in Eq. 6. The elevation of the subgrade at the point of control must be such that the energy gradient at that point (obtained by adding the depth and the velocity head to the elevation of the subgrade) is equal to the energy gradient at the downstream end of the crest, minus intervening losses. Having located the control, the hydraulic computations may be completed and the profile platted.

(*g*) *Comparative estimates.* As soon as a bottom profile has been completed, the principles of Fig. 12 may be applied to several sections of the spillway

crest, and the resulting channel platted on a topographic map of the site. A cost estimate for the spillway may then be prepared which will locate *one* point on one of the curves in the left-hand part, (a), of Fig. 14. By assuming additional trial values of a, with the same value of n, and making more cost estimates, a number of points may be computed from which one of the curves in Fig. 14a may be drawn. The lowest point on this curve will give the correct value of a for use with the trial value of n. A new value of n should then be assumed, and the process repeated, platting other curves as shown in the diagram until it is clear that the most suitable combination of a and n has been found. The curve in Fig. 14b is obtained by platting the lowest point of each of the curves in Fig. 14a against their respective values of n.

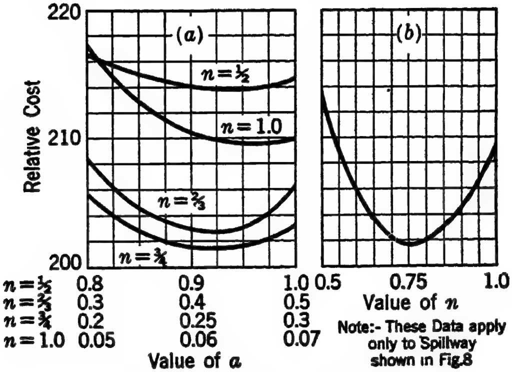


FIG. 14. Economic cost curves.

In the example given it is seen that 0.75 is the best value for n, the corresponding value for a being approximately 0.25.

(h) *Submergence of crest.* The amount the weir can be submerged at B, Figs. 9 and 13, without seriously reducing the discharge, has an important bearing on economy. This is apparent from the fact that the bottom profile is obtained by measuring $(d + y)$ downward from the permissible submergence line BP, Fig. 9. The effect of submergence on discharge is discussed in Art. 3(m) of Chapter 11. Submergence reduces the discharge only slowly up to a considerable depth, and the backwater depth reduces downstream from point B. The crest may be submerged at B by one-half or two-thirds of the head without seriously reducing the total discharge. Eqs. 6 and 7 are not strictly applicable to such a partly submerged condition, as the inflow is variable. However, they apply with sufficient accuracy to any ordinary case, and the results may be checked by methods to be given subsequently. It may be desirable sometimes to assume a heavy submergence, increasing the depth of spill to compensate for the reduced discharge factor. The limit to which this process may be carried can be determined by trial estimates.

(i) *Allowance for swell and turbulence.* In estimating the freeboard required to prevent slopping over the edges of the channel or the top of the lining,

allowance must be made for turbulence and "swell" due to unequal distribution of velocities and entrained air drawn into the stream by the infalling water. Numerical data on the amount of swell are scarce. Experiments by Hinds [3] on a small-scale model of the spillway of Fig. 8 indicated swells from zero to 10 per cent, averaging about 4 per cent. In addition to swell, cross-flow causes a surge up the outside bank of the channel. For the spillway of Fig. 8 these effects were not important as the height of the back of the channel was everywhere in excess of requirements.

Downstream from the point of inflow the swell tends to decrease, as the entrained air escapes, but unbalanced flow may cause a zigzag eddy to persist for a considerable distance down the channel. There are some conditions under which this may be important.

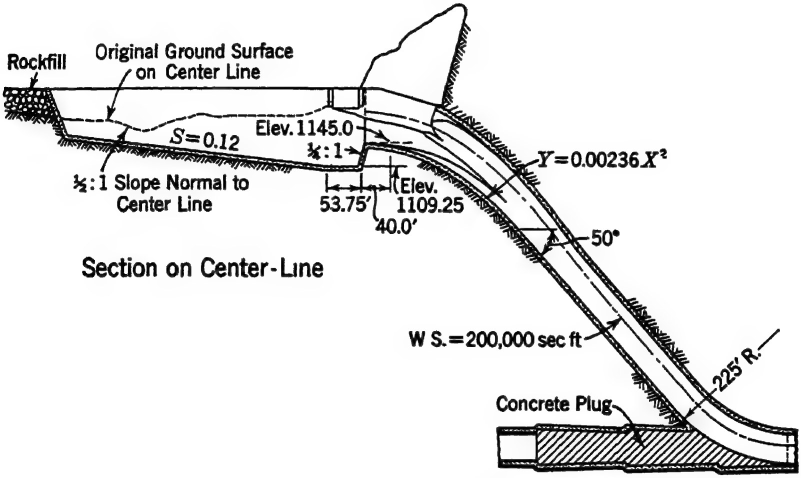


FIG. 15. Section on center line, Boulder Dam spillway.

(*j*) *Boulder Dam spillways.* At Boulder Dam, two side-channel spillways are provided, each having a capacity of 200,000 cu ft per sec at maximum flood. A section through one of these is shown in Fig. 15. Here it was necessary to make careful allowance for swell at the tunnel inlet and to design the transition to minimize cross-flow within the tunnel. Exhaustive model tests were carried out to these and other ends.[4]

As shown in Fig. 15, the entrance to the tunnel is raised above the channel floor. Also, to improve flow within the tunnel, the inlet was offset in plan, leaving a shoulder on the crest side of the portal. The profile of the curving tunnel invert just below the portal was determined from the trajectory of the jet, using the principles applicable to overflow crests as explained in Chapter

[3] Op. cit.

[4] Boulder Canyon Project Final Reports, Part 6, Hydraulic Investigations, Bull. 1, Model Studies of Spillways, U. S. Bur. Reclamation, Denver, Colo., 1938.

11. The form of the structure was determined by trial and checked by hydraulic model tests.

(*k*) *Checking existing structure.*

(1) *Hydraulic formula.* The information already given is intended for use in the preparation of new designs not hampered by special limiting conditions. The same fundamental hydraulic principles may be applied to the analysis of an existing channel, to the design of a channel which must conform to certain prescribed dimensions and grades, or where inflow is non-uniform. The fundamental momentum equation can be written to make it applicable, approximately, to finite values of Δx, thus:[5]

$$\frac{\Delta M}{\Delta x} = \frac{Q\Delta V}{g\Delta x} + \frac{q}{g}(V + \Delta V) \qquad [10]$$

from which may be derived the equation

$$\Delta y = \frac{Q_1}{g}\frac{(V_1 + V_2)}{(Q_1 + Q_2)}\left[\Delta V + \frac{qV_2\Delta x}{Q_1}\right] \qquad [11]$$

where Δy is the drop in water-surface curve in the reach of length Δx, ΔM and ΔV are the corresponding changes in momentum and velocity, Q_1 and V_1 are discharge and velocity at the upper end of the reach, Q_2 and V_2 the same at the lower end, q is inflow per foot of crest, and g is gravity.

Once a starting point is found, the application of Eq. 11 to successive short reaches of the channel produces a water-surface curve. This is done by trial. Having selected a reach of length Δx, in a specific location, Q_1 and Q_2 are first determined. If the depth at one end is known, the velocity at that end can be computed. A trial depth is then assumed for the other end, from which the remaining velocity may be computed. The values thus obtained are inserted in Eq. 11 to obtain a value for Δy. This value, plus estimated friction, should equal the difference in elevation of energy gradients at the two ends, as obtained by adding depth plus velocity head to flood elevation. If not, assume a new trial depth and repeat the computation.

(2) *Location of control.* Before Eq. 11 can be applied, a starting place at which the velocity is known must be found. Such a starting place will be located at a point of control where the depth passes through the critical point from above to below. Its location is not as simple as in the case of a new design, as previously described.

If the channel slope between D and N, Fig. 8, is insufficient to support flow at the critical depth, and below N more than sufficient, the control will be located at N. If the slope between D and N is greater than required to support flow at the critical depth, the control will come at D or at some point above. Upstream from D there is, in addition to the force of friction,

[5] JULIAN HINDS, "Side Channel Spillways," *Trans. Am. Soc. Civil Engrs.*, Vol. 89, 1926, p. 881.

a resisting force due to inertia of the incoming water, which affects the control. If the slope of the channel immediately upstream from D is insufficient to overcome losses and maintain flow at the critical depth, the control will be at D; otherwise the control will come at some point farther upstream where the slope is just sufficient for this purpose.

The actual location of a control above D is complicated by the fact that the critical depth, the inertial resistance, and the discharge are all variable. A suggested method of attacking the problem is illustrated in Fig. 16. The

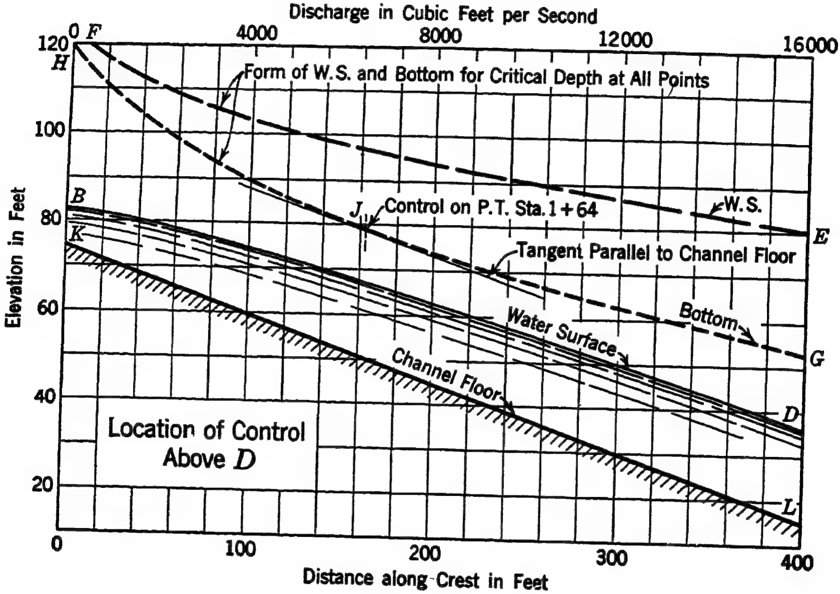


FIG. 16.

channel is divided into a number of reaches and the discharge and critical depth are computed at the end of each reach. Inertial resistances and friction losses that would occur if the flow were *everywhere* at critical depth are estimated. Beginning at some fictitious elevation opposite the downstream end of the crest, as at E, Fig. 16, and accumulating friction losses and inertial resistances, it is possible to plat the curve EF, which the water surface would need to follow if the depth were critical throughout (with the fictitious end elevation of E). Measuring critical depths down from this curve gives the bottom profile HG that would be necessary to produce this condition. The control is at point J, where the tangent to the curve HG is parallel to a tangent to the actual bottom KL at the same station. The slope required to maintain critical flow to the left of J is greater than the actual slope, and to the right it is less, which is the condition necessary to the formation of a control. It is not necessary that KL be straight. It is possible to have two or more control points with hydraulic jumps between.

(*l*) *The backwater curve.* Having located a control, the backwater curve may be calculated both ways, as explained in Art. 5, Chapter 11, and including an allowance for inertial resistances computed from Eq. 11.

(*m*) *Channel below spillway.* For miscellaneous structural details and the design of the chute from the side channel spillway to the river, see Art. 1.

3. Shaft Spillways. A shaft spillway, sometimes termed a "morning glory spillway," consists of a vertical flaring funnel, as shown in the accompanying

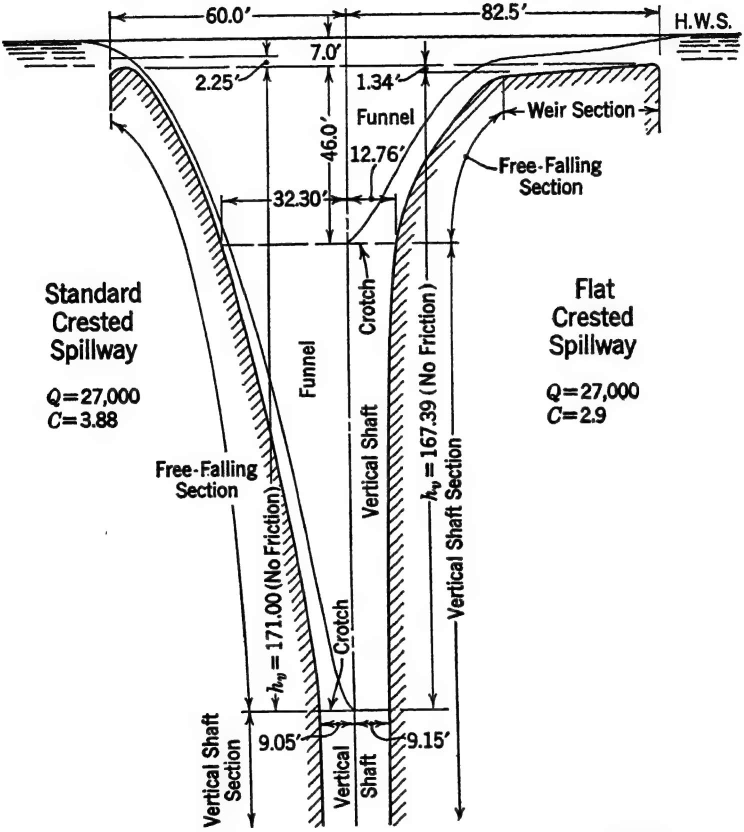


Fig. 17. Comparison of standard with flat-crested shaft spillway.

illustrations, with its top as the lip of the spillway. The funnel connects with an ell shape outlet conduit extending through or around the dam.

There are two general types of shaft spillways, the first having a standard crest (see Art. 2, Chapter 11) and the second a flat crest. These are compared in Fig. 17. The flat-crested spillway consists of a "weir section," a "free-falling section," where the shape of the spillway conforms to the path of a free-falling jet, and a "vertical shaft section," which is completely filled with water. Below the vertical shaft section is the elbow and horizontal conduit.

In the standard-crested spillway the outline is the same except that there is no weir section, the water beginning its free fall immediately upon leaving the crest, whereas in the flat-crested spillway the water is caused to approach the crest on a flat slope before beginning its free fall.

The standard-crested spillway has the advantage of a smaller diameter crest, since its coefficient of discharge is greater than that of a flat crest, the former diameter being only about 75 per cent of the latter. Other requirements being met, the standard-crested spillway therefore is more advantageous in case the spillway is a tower, as in Fig. 20.

However, the flat-crested spillway has the smaller funnel, the diameter of that shown in Fig. 17 at a distance of 46 feet below the crest being only about 39 per cent of that required for the other type. Therefore it is always to be preferred where the spillway is excavated in the rock, as in Fig. 23.

The vertical shaft section of the standard-crested spillway is always somewhat smaller in diameter since the head available for vertical velocity, h_v, is somewhat greater. However, this difference is usually negligible.

The height of the free-fall section (Fig. 17) may be greater than the available vertical distance to the elbow. In this case, even though the spillway is a tower, the flat-crested or a composite spillway may have to be adopted or else the head on the spillway increased, resulting in a smaller diameter, and the reservoir level maintained by gates on the crest.

This article will treat of the two general types of spillways. The background being laid, the reader will have no trouble in designing the composite type.

(*a*) *The standard-crested spillway.* Fig. 18 shows a section of a standard-crested spillway, used as an explanatory example. The reader is referred to Art. 2 of Chapter 11 for a description of how the "standard crest" is obtained by simply filling in with concrete the space under the jet from a sharp-crested weir. The shape of the standard-crested shaft spillway is found by the same general principle except that the crest is circular and the jet partakes of special shapes.

Experiments [6] have been made to determine the discharge capacity and the shape of the lower nappe for a circular, vertical, sharp-crested spillway with various ratios of head, H, on the sharp crest to the radius, R, of the sharp crest. These experiments were conducted with negligible velocity of approach. No data are available for appreciable velocity of approach.

The author has used data contained in the reports of these experiments for the design of the standard-crested shaft spillway. Where references are not given, the author's judgment and extensions of the data have been used.

Reference is made to Fig. 18, which records the following steps in the design. It will be noted that the design of the "spillway crest" is predicated on the flow over a theoretical "sharp crest," as noted in the figure.

[6] CAMP and HOWE, "Tests of Circular Weirs," *Civil Eng.*, April 1939, p. 247. R. B. DUPONT, "Determination of Under Nappe over a Sharp-Crested Weir, Circular in Plan with Radial Approach," Thesis submitted to Case School of Applied Science, 1937.

(1) The required capacity, Q, and the allowed maximum head, h, on the spillway crest, under which the spillway must operate, are given. For this example, assume

$$Q = 30{,}000 \text{ sec ft}$$

$$h = 10.00 \text{ ft}$$

(2) Assume a *tentative* head, H, on the sharp crest of 11.2 ft.

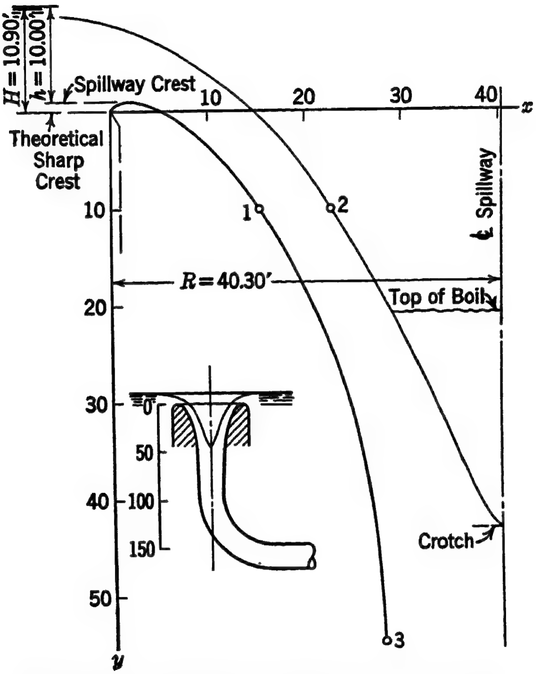


FIG. 18. Example of standard-crested shaft spillway.

(3) Assume a *tentative* value of R, the radius of the sharp-crested weir of 50.0 ft. Then

$$\frac{H}{R} = \frac{11.2}{50} = 0.224$$

(4) From Fig. 19, derived by the author from data in duPont [7] find that for $\dfrac{H}{R} = 0.224$, the coefficient of discharge $C = 3.311$.

(5) Then find the discharge, q, per linear foot of crest.

$$q = CH^{3/2} = 3.311 \times 11.2^{3/2} = 124.1$$

[7] *Idem.*

(6) The required radius, R, is then

$$R = \frac{Q}{2\pi q} = \frac{30{,}000}{2 \times 3.141 \times 124.1} = 38.5 \text{ ft}$$

(7) Since this value of $R = 38.5$ is not equal to the value of $R = 50.0$ assumed in item 3, a new value of R must be assumed for item 3 and the process repeated until the value of R obtained in item 6 equals that assumed in item 3.

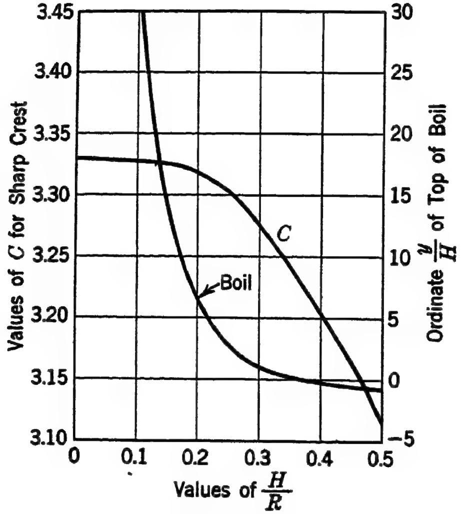


FIG. 19.

Thus, assuming a correct value of R in item 3 of 38.8 ft, we find that

$$\frac{H}{R} = 0.2887$$

$$C = 3.282$$

$$q = 123.2$$

$$R = 38.8$$

(8) We have now obtained results for our tentative value of $H = 11.2$ ft in item 2 and we must determine if this is correct.

(9) The rise, r, of the lower nappe, after it leaves the sharp crest, is given by the following equation from Camp and Howe.[8]

$$\frac{r}{H} = 0.11 - \frac{0.10H}{R}$$

$$r = 0.11 \times 11.2 - \frac{0.10 \times 11.2 \times 11.2}{38.8} = 0.909$$

[8] *Op. cit.*

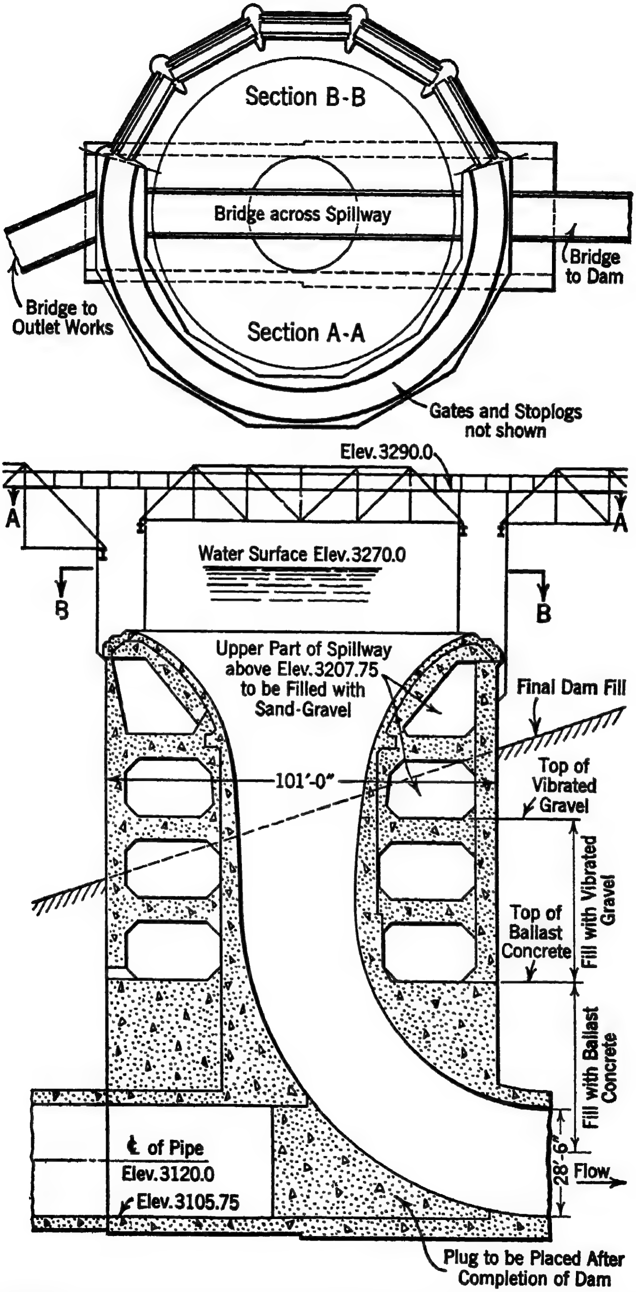


FIG. 20. Kingsley Dam shaft spillway (*Central Nebr. Public Power and Irrigation District*)

(10) Then the head, h, on the spillway crest is

$$h = H - r = 11.2 - 0.909 = 10.29$$

Since this value of h is not equal to the value of $h = 10.00$ according to item 1, a new value of H must be assumed in item 2 and the entire process repeated until the final value of h in item 10 is equal to 10.0.

Thus, assuming a correct value of $H = 10.9$ in item 2 and a correct value of $R = 40.3$ in item 3, find that

$$\frac{H}{R} \doteq 0.2703$$

$$C = 3.294$$

$$q = 118.6$$

$$R = 40.3$$

$$r = 0.90$$

$$h = 10.0$$

(11) The next step is to plot the jet flowing over the sharp-crested weir. This is obtained from data contained in duPont's thesis except for the maximum rise, r, of the jet, which is taken from data in Camp and Howe, as previously explained. The coordinates of the under nappe, referred to the sharp-crested weir as origin, are given in Fig. 21.

For Point 1, where $y = 10.0$

$$\frac{y}{H} = \frac{10.0}{10.9} = 0.918$$

From Fig. 21, for $\frac{H}{R} = 0.2703$, $\frac{x}{H} = 1.4$

Then $x = 1.4 \times 10.9 = 15.27$

(12) Values of x_0 for the outside of the jet, below the sharp crest, can be obtained from

$$x_0 = R - \sqrt{R_0^2 - \frac{Q}{\pi\sqrt{2g(y + 0.387H)}}}$$

where R_0 is the radius of the lower nappe at that elevation. In this equation, $(y + 0.387H)$ is the head available for vertical velocity. For Point 2, $R_0 = R - x = 40.30 - 15.27 = 25.03$ and $y = 10.0$

and $x_0 = 40.3 - \sqrt{25.03^2 - \dfrac{30{,}000}{3.141 \times 8.02\sqrt{10 + 0.387 \times 10.9}}}$

$x_0 = 22.66$

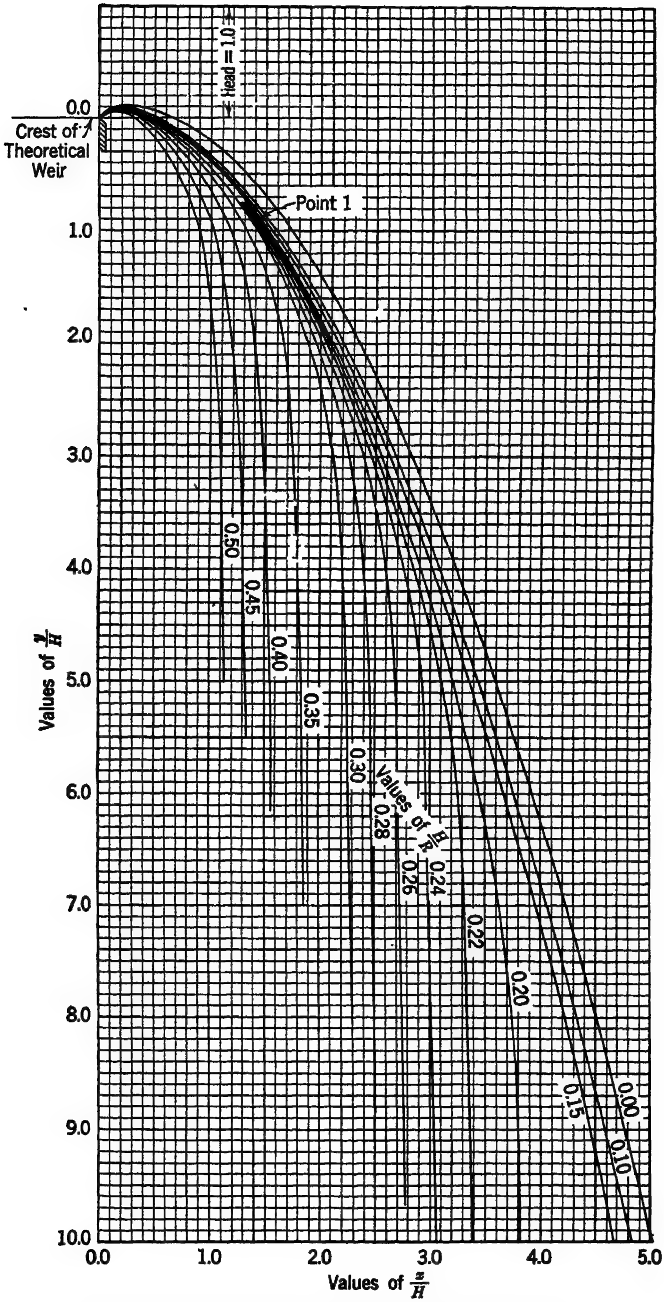


FIG. 21. Under nappe for standard-crested spillway.

(13) Proceeding in this way we find that the upper nappe intersects the center line of the spillway when $x_0 = 40.30$ and $y = 43.20$.

At this point, called the "crotch," the horizontal velocity ceases, its energy being converted into a "boil," which occurs above the crotch as

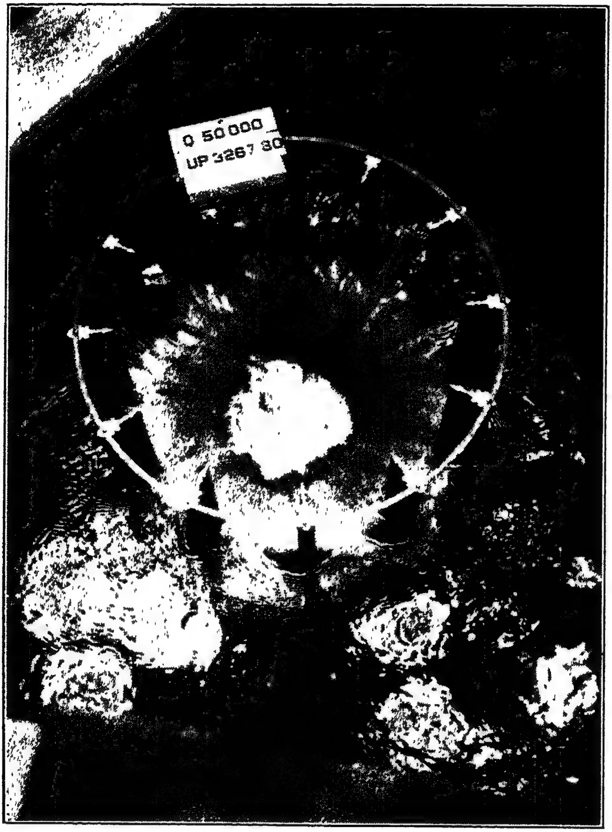


FIG. 22. Kingsley Dam shaft spillway test (*made by G. E. Barnes, Case School of Applied Science*).

shown in Figs. 18 and 22. The ordinate $\frac{y}{H}$ of the top of the boil is shown in Fig. 19 (duPont) which, since for this example $\frac{H}{R} = 0.2703$, locates the boil at $y = 1.9 \times 10.9 = 20.7$ ft below the sharp crest.

Below the elevation of the crotch we proceed as follows:

(*b*) *The discharge conduit.* Thus far we have neglected friction since, for spillways of the proportions indicated in Fig. 18, the friction loss from the crest to the crotch is small and, since the neglect of it is well within the accuracy

of the problem and is difficult to take into consideration, it may be neglected. However, below the crotch there must be a conduit consisting of a vertical shaft, an elbow, and a horizontal or nearly horizontal member in which the friction loss will be considerable.

The diameter of the vertical shaft below the crotch should continue to decrease until the required size of the conduit is reached. Before the required size of the conduit is reached, the diameter of the vertical shaft can be obtained as follows.

Let R_1 be the radius of the shaft at a given elevation and h_v the available velocity head at that elevation. Then

$$h_v = y + 0.387H - f_1 - f_2$$

where f_1 is the friction loss from the crest to the crotch and f_2 is the friction loss from the crotch to that elevation. These can be determined by the ordinary principles of hydraulics. Then

$$Q = \pi R_1^2 \sqrt{2g(y + 0.387H - f_1 - f_2)}$$

or

$$R_1 = \sqrt{\frac{Q}{\pi\sqrt{2g(y + 0.387H - f_1 - f_2)}}}$$

For Point 3, $y = 55$. Assume that $f_1 = 0.3$ and $f_2 = 0.4$. Then $R_1 = 12.48$.

As mentioned above, the vertical shaft can decrease in this manner until the proper size of the conduit is reached. The proper size of the conduit is that size in which the discharge, Q, can be carried according to the head available. If, for instance, the proper size has been reached at Point 3, and since all of the head above Point 3 has been used to create velocity and overcome friction and other losses, the remaining head available to overcome friction is equal to the difference in elevation between Point 3 and the outlet of the conduit.

However, P. H. Jaenichen has pointed out that, at the elbow, there is one feature of great importance in shaft spillways which may have been neglected in some cases.

Let p = the average pressure at a given cross-section of the elbow,
p_a = atmospheric pressure, about 34 ft of water at sea level,
p_c = the reduction of pressure on the inside of the elbow, due to centrifugal force,
p_v = vapor pressure of water, about 1 ft of water (see steam tables).

The absolute pressure on the inside of the bend is then equal to

$$P = p_a + (p - p_c)$$

In a model test of a spillway, p_c frequently is greater than p so that the piezometer reading, $p - p_c$, is negative. However, in the model, this negative value of $p - p_c$ is seldom if ever sufficient to reduce the absolute pressure, P, to the vapor pressure of water, p_v.

But, when the model is stepped up to the prototype, $p - p_c$ also steps up in proportion to the scale model but p_a of course remains constant. Therefore, as the negative value of $p - p_c$ increases while p_a remains constant, the absolute pressure, P, decreases and may be equal to or less than the vapor pressure of water. If this should occur, serious consequences would ensue. In addition to serious cavitation, the elbow would not remain full of water, the inside being filled with water vapor. If the discharge remains constant, the velocity in the elbow must increase, due to reduced section occupied by the water, and a greater pressure would be required to produce this greater velocity.

If the pressure, p, is not sufficient to provide for the greater velocity and friction, the capacity of the spillway will be greatly reduced.

Since the reduction in pressure on the inside of the elbow cannot be calculated closely, model tests are necessary for the final design. The exact conditions at the elbow can be simulated by subjecting the entire model to a partial vacuum in order to make the ratio of the absolute pressure surrounding the model to atmospheric pressure equal to the scale ratio of the model. This means, of course, that to prevent separation of the jet, the scale ratio of the model cannot be greater, theoretically, than $1/(p_a - p_v)$ or about 1/33, and, practically, somewhat less, depending upon the facilities of maintaining constantly the partial vacuum.

As mentioned before, the shaft must have a section of constant diameter of sufficient length to accumulate the necessary pressure to balance the tendency for subnormal pressures in the elbow. The proportions of some spillways are such that, the elevation of the elbow being fixed, the elevation of the crotch is too low if a standard-crested spillway is used. If this is the case, the flat-crested shaft spillway or a composite type must be adopted.

(c) *The flat-crested shaft spillway.* The design of a flat-crested spillway has been described by Ford Kurtz [9] and will only be summarized here, with certain noted variations recommended by the author. The reader is referred to the foregoing description of the standard-crested spillway for fundamental principles.

The necessary steps required for the design of a flat-crested spillway are as follows, reference being made to Fig. 24, which represents the conditions of the Davis Bridge spillway of Fig. 23.

(1) The required capacity, Q, and the allowed maximum head, h, under which the spillway may operate are given. For this example

$$Q = 27{,}000 \text{ sec ft}$$

$$h = 7 \text{ ft}$$

(2) The discharge per linear foot is given by the equation

$$q = Ch^{3/2}$$

[9] FORD KURTZ, "The Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam and Hydraulic Tests on Working Models," *Trans. Am. Soc. Civil Engrs.*, 1925, p. 1.

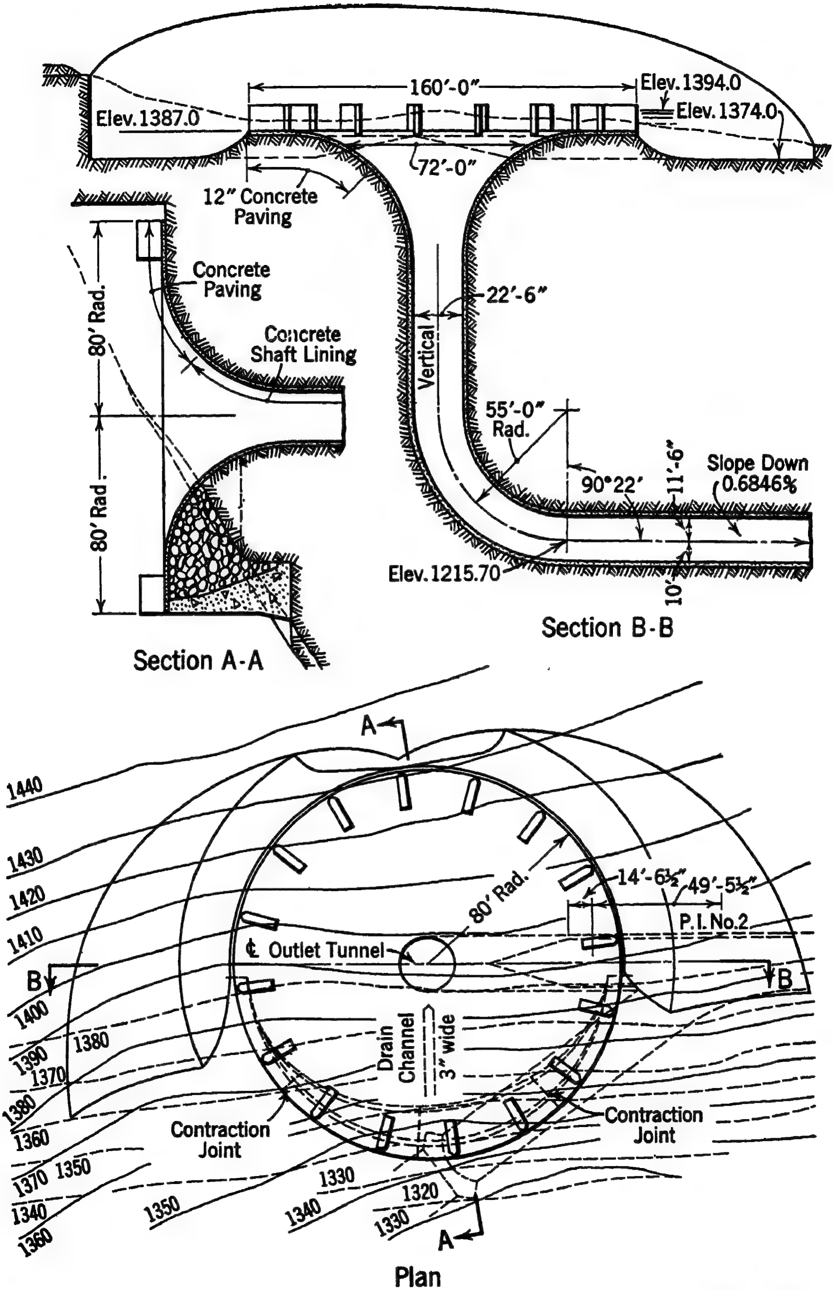


FIG. 23. Davis Bridge Dam shaft spillway (*Eng. News-Record*, Jan. 24, 1924, p. 148).

Since this is to be a flat-crested spillway, the theoretical value of C is 3.087. Practical values for several types of flat-crested spillways are given in Water-Supply Paper 200, U. S. Geological Survey. A value of $C = 2.9$ was adopted for the Davis Bridge spillway where the ratio of the radius of the rounded upstream corner to the head on the crest is $2.5/7 = 0.36$. Therefore $q = 2.9 \times 7^{3/2} = 53.7$ sec ft

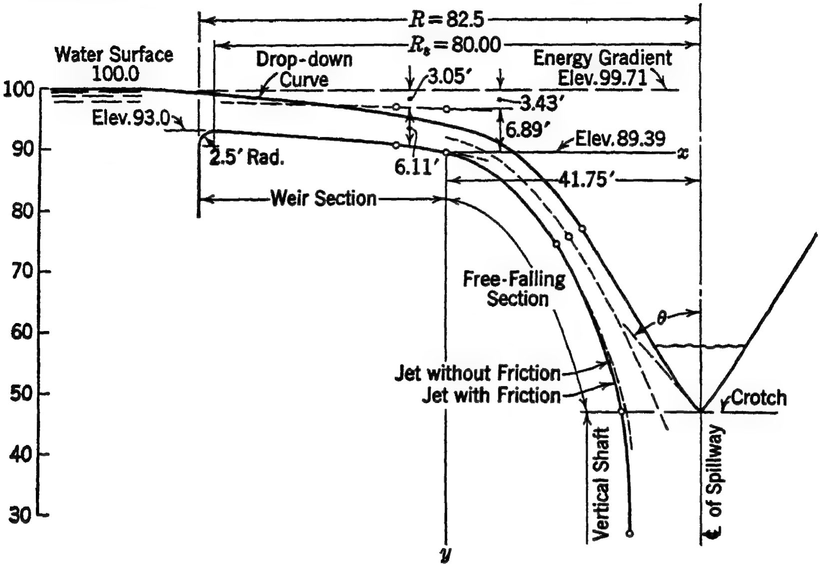


FIG. 24. Example of flat-crested spillway.

(3) The required length of crest is

$$l = \frac{Q}{q} = \frac{27{,}000}{53.7} = 503 \text{ ft}$$

(4) And the required radius is

$$R_s = \frac{l}{2\pi} = \frac{503}{2\pi} = 80.0 \text{ ft}$$

For the Davis Bridge spillway, this radius was taken as the outside radius of the spillway but the author prefers to use it for the summit of the spillway, as shown in Fig. 24. Thus the outside radius of the spillway is

$$R = R_s + r = 80.00 + 2.5 = 82.5 \text{ ft}$$

(5) The loss of head, h_f, at the upstream edge of the weir is as follows·

$$Q = Ch^{3/2} = 3.087(h - f)^{3/2}$$

From which

$$f = h\left[1 - \left(\frac{C}{3.087}\right)^{2/3}\right]$$

For $h = 7$ and $C = 2.9$

$$f = 0.29 \text{ ft}$$

(6) Assuming the water surface in the reservoir to be at el 100.00, the elevation of the energy gradient in the spillway, neglecting all other losses, is

$$100 - f = 100.00 - 0.29 = 99.71$$

This is plotted in Fig. 24.

(7) We are dealing with the principles of flow involved in a flat-top weir. However, since the circumference of the spillway is constantly reducing toward the center line, the floor of the weir must drop toward the center line to make the same principles hold. To find the elevation of the floor for any radius R_1, measure down from the energy gradient a distance equal to

$$d = (h - f)\left(\frac{R_s}{R_1}\right)^{2/3}$$

If $R_1 = 50$,

$$d = (7 - 0.3)\left(\frac{80}{50}\right)^{2/3} = 9.16$$

and the floor of the weir is at el $99.71 - 9.16 = 90.55$. Thus the floor of the weir can be plotted.

(8) The depth of water at any place can be obtained from

$$h_1 = \frac{2d}{3}$$

Thus for $R_1 = 50$ and $d = 9.16$, $h_1 = 6.11$, and the water surface can be plotted. The "drop-down curve" at the crest can be sketched in, assuming that it extends about $2h$ beyond the upper end of the weir.

The "weir section" of the spillway, thus determined, must be terminated at some limiting value of R_1 and allowed to continue as a freely falling jet. It is difficult to decide on the limiting value of R_1 for any particular problem. It is desired to make the crotch as high as possible, for reasons previously explained, and this calls for a low value of R_1. On the other hand, the angle, θ, of inclination of the upper surface of the jet with the vertical at the crotch, must not be too large, as this will result in too great a disturbance when the jets come together. The end of the weir section is usually located where the neglect of the vertical component of the velocity will result in negligible error and where the freely falling section may be assumed to become horizontal.

A limiting value of R_1 of 41.75 ft, adopted for the Davis Bridge spillway, will be used in this example. From the equation of item 7, the elevation of

the bottom of the weir is $d = 10.32$ ft below the energy gradient, the depth of water (neglecting the drop-down curve explained later) is $h_1 = 6.89$, and the water surface is $d - h_1 = 3.43$ ft below the energy gradient, this latter being the velocity head.

Again neglecting the drop-down curve, the tangent, S, of the average slope of the filaments of water at that place is 0.1. Therefore the vertical velocity is $S = 0.1$ times the horizontal velocity and the vertical velocity head is approximately $S^2(d - h_1) = 0.01 \times 3.43 = 0.0343$, the neglect of which is well within the accuracy of the problem.

(9) With the bottom of the adopted end of the weir section as the origin of coordinates x and y, as shown in Fig. 24, Kurtz [10] has determined that the center line of the freely falling jet is given by the following equation. However, duPont and Camp and Howe [11] indicate that the center line of the jet may be somewhat steeper than theory indicates. Therefore this equation must be considered only a close approximation to be checked by tests as later recommended.

$$y + 0.36h_1 = \frac{(x + 0.36h_1)^2}{4.56h_1}$$

where h_1 is the depth of water at the adopted end of the weir section (neglecting the drop-down curve) as determined in item 8.

For $x = 20$ ft and $h_1 = 6.89$ as previously determined, this equation gives $y = 13.59$.

(10) The thickness of the jet at any point on its center line is given by Kurtz as

$$t = \frac{Q}{2\pi R_1 \sqrt{2g(y + 1.5h_1)}}$$

This thickness is laid off normal to the jet center line, with one-half on each side until the crotch is reached.

For $x = 20$, $R_1 = 41.75 - 20 = 21.75$, $y = 13.59$, and $h_1 = 6.89$ Then the above equation gives $t = 5.04$.

The drop-down curve can then be approximated by eye.

(11) Thus far we have neglected the effect of friction, with the exception of the loss, f, considered in item 5 for loss at entrance. An analysis of the hydraulic characteristics will show that for this weir section, the friction loss was about 0.02 and, for the free-falling section, 1.04 ft for this example, making a total of 1.06 ft. Since this is well within the accuracy of the problem, it may be neglected in determining the shape of the jet. However, this loss and subsequent friction losses should be taken into consideration for the remainder of the spillway.

[10] Op. cit.

[11] Op. cit.

(12) The size of the vertical shaft at any point below the crotch is determined as follows:

Zero vertical velocity is at the vertex of the parabolic center line of the free-falling jet, the equation of which is given in item 9. From this equation, it is seen that the vertex is where $y = -0.36h_1$. Therefore the total vertical velocity head is $y + 0.36h_1$. The net vertical velocity head is

$$h_v = y + 0.36h_1 - f_1 - f_2$$

where f_1 and f_2 are the friction losses from the crest to the crotch and from the crotch to the point in question, respectively.

Therefore the required radius, R_1, of the vertical shaft below the crotch is determined as follows:

The velocity is $v = \sqrt{2g(y + 0.36h_1 - f_1 - f_2)}$. The area of the section is $A = \pi R_1^2$. Also $Q = Av$. Therefore

$$R_1 = \sqrt{\frac{Q}{\pi\sqrt{2g(y + 0.36h_1 - f_1 - f_2)}}}$$

For $y = 42.00$, the scaled elevation of the crotch, and letting $f_1 = 1.06$, as indicated before, and $f_2 = 0$ for this elevation, we find that $R_1 = 12.76$.

It is seen that this radius is slightly larger than the dimension calculated for the dot and dash free-falling jet, since the latter neglected friction. Therefore the under nappe of the free-falling jet can be adjusted by eye as indicated by the full line in the figure.

It should be noted that it is sometimes assumed that a portion of the vertical velocity head is lost at the crotch. Kurtz assumed arbitrarily that $0.36h_1 = 0.36 \times 6.89 = 2.48$ ft was lost. However, the author sees no need for assuming any loss of the vertical velocity head, other than that noted above for pure skin friction. Deducting the 2.48 ft of loss assumed by Kurtz results in a radius at the crotch of 12.92 ft instead of the 12.76 calculated above.

For $y = 62$ and letting $f_1 = 1.06$, as indicated before, and $f_2 = 0.56$ ft, as determined by trial and error, we find that $R_1 = 11.62$.

The vertical shaft can decrease in this manner until the proper size of conduit is reached, as explained in detail in section (*b*) *the discharge conduit.*

(*d*) *Discharge capacity of shaft spillways.* Fig. 25 shows the rating curve of the shaft spillway of the Kingsley, Neb., Dam, as determined by model test. Curve AA' is the rating curve of the crest of the spillway without interference from backwater in the shaft. This curve is no different from that of any standard-crested plain spillway dam.

Curve BB' is the rating curve of the shaft and horizontal conduit. Since the head acting on the shaft and horizontal conduit is large in comparison with the head on the spillway crest, changes in the elevation of reservoir water surface make relatively little difference in its capacity.

Up to about 54,000 sec ft the crest rating curve governs and above about 54,000 sec ft the shaft rating curve governs. Complaint has been made that, above a given discharge, represented by point C, this type of spillway has no reserve capacity. For instance, if the freeboard of the dam had been provided for a maximum head on the crest of 15 ft, a reduction of this freeboard by 3 ft would increase the discharge from 54,000 to 55,200, or 2 per cent; whereas, if a standard-crested *plain* spillway had been used, this 3-ft encroachment on the freeboard would have increased the discharge from 54,000 to 68,000 sec ft, or 26 per cent.

However, this apparent defect is due to restricted discharge channels rather than to the type of spillway, since in the side channel spillway and other types of spillways, the same conditions apply. Even for the ordinary low overflow spillway dam, tailwater sometimes submerges the crest and restricts discharge capacity in exactly the same manner. Thus it is apparent that an enlargement of the outlet conduit will remove this restriction in capacity; that is, design the spillway for a greater head and corresponding greater capacity.

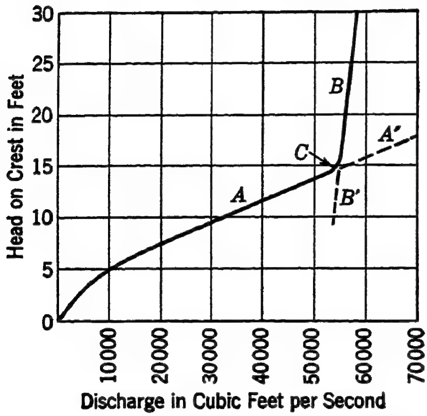


FIG. 25. Rating curve of Kingsley Dam shaft spillway (*G. E. Barnes, Case School of Applied Science*).

(*e*) *The effect of piers on the crest.* Radial piers on the crest to support gates or flashboards are used frequently on shaft spillways, as indicated in the accompanying illustrations. Piers are also required in order to prevent spiral flow in the funnel of the spillway. Where such piers are used, their relation to discharge capacity of the crest can be obtained from Art. 3 of Chapter 11.

It is difficult to evaluate the effect of such piers on the design of the spillway, and model tests are required for the final design. For preliminary designs, it is sufficient to follow the foregoing methods, but to design for a discharge, Q', determined as follows:

$$Q' = Q\frac{l + nt}{l}$$

where Q = the required discharge,
l = the length of crest required without piers,
n = the number of piers,
t = the thickness of piers plus contractions.

(*f*) *Model tests.* Possible accuracy with present methods of design of shaft spillways is not sufficient to proceed without the assistance of model tests. Also it is necessary to determine by such tests the location and amount of

negative pressures which may obtain not only for the design head but for lower heads.

It is suggested that, for a standard-crested spillway, a preliminary test be made with a circular sharp-crested weir (du Pont, op. cit.) equipped with piers if required. The shape of the jet having been determined, the model can be completed and the final test made.

For a flat-crested spillway, the preliminary test would be conducted by installing only the weir section (Fig. 17) and measuring the shape of the under nappe.

4. Emergency Spillways. All engineers agree that an emergency spillway is one which will be called upon to operate so infrequently that it is not considered necessary to protect the spillway control, the structure, its foundation, or its discharge channel from serious damage when it goes into action. However, further definitions of this type of spillway vary according to two points of view.

Before we discuss these two points of view, assume that the "spillway design-flood," as defined in Art. 50*d* of Chapter 5, is the flood which, after a thorough study of the drainage area, it has been decided must be carried past the dam without failure of that structure. This study, we will say, has been made by a competent hydrologist who has no knowledge of how the designers intend to take care of it.

In the consideration of an emergency spillway, one must have at least some understanding of the probable frequency of occurrence of such a flood. We have no means of estimating this exactly, but it is agreed that the probable frequency of such a flood, or even 60 to 80 per cent of it, is not measured in hundreds but in thousands of years.

Thus the probability of 60 to 80 per cent of such a flood occurring is too remote for economic consideration, provided that a calamity can be prevented.

From the first point of view, the emergency spillway is an auxiliary spillway which would be called into action should a flood greater than the spillway design flood occur. From that viewpoint it is simply an added factor of safety.

From the second point of view, the emergency spillway is an auxiliary spillway which would be called into action should a flood occur whose magnitude was 60 to 80 per cent of that of the spillway design flood.

These two points of view would coincide if it could be granted that from the first point of view the estimate of the spillway design flood is inadequate, while from the second point of view it is completely adequate. Then, in both cases, the permanent spillway would be capable of taking only a percentage of the proper flood to be controlled and the emergency spillway the balance. That is, a permanent spillway would be built to accommodate 60 to 80 per cent of the proper flood and an emergency spillway would be provided to take the balance with considerable damage to it but with safety to the dam.

The damage caused by the operation of an emergency spillway may vary from the loss of the control apparatus to the complete washing out of the

spillway structure and its foundation, even to the extent of emptying the reservoir, but at such a low rate of discharge as to cause a downstream flood which would be insignificant compared with that resulting from the loss of the dam.

The emergency spillway has its parallel in the emergency channels of the Mississippi River, where a section of levee is purposely left low or is ruptured in places where severe but relatively small damage would occur in order to prevent otherwise certain rupture of a levee system protecting a large city.

An emergency spillway is most easily obtained at a low divide in the reservoir rim. If the elevation of the divide is so low that a dike is required, the

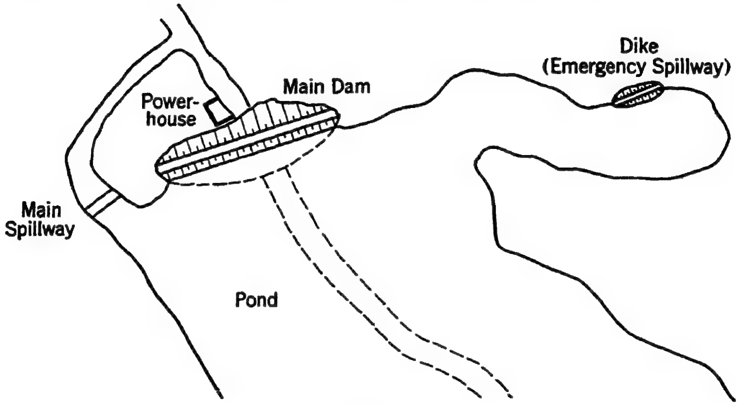


FIG. 26.

dike is left at such an elevation that it will be overtopped before the main dam is overtopped.

A typical emergency spillway is shown in Fig. 26. The elevation of the top of the small dike, located at a low divide, is placed at the maximum elevation of water surface corresponding to safety for the dam, the top of the dam being of course at a higher elevation to accommodate waves and to provide that, should a greater rise of water surface occur, the dike would be washed out first.

An ideal site for such an emergency spillway is where there is a dip or surface syncline in the bedrock close to the surface so that, should the dike and the earth overburden be washed out, the scour would be limited to the rock surface but the resulting discharge would be sufficient to save the dam.

Under such conditions, the flow from the emergency spillway might scour out the discharge channel leading to the river, fill up the tailrace of a power plant and perhaps do other damage.

However, this type of emergency spillway has also been provided where no bedrock is present and where the failure of the dike would scour the foundation even possibly to the extent of emptying the reservoir, but, owing to the relatively slow erodibility of the material and the great width of the divide on which the dike is built, at a rate which would be so slow as not to cause a calamity downstream.

The dike of the emergency spillway shown in Fig. 26 is well protected from wave action. Where this is not the case, it must be protected by a breakwater, since the elevation of water surface at which it would fail must be fixed closely.

Should there be no low place in the divide requiring a dike, an emergency spillway can be provided if a channel cut through a divide is not too expensive. In such cases, the channel should be cut to a depth sufficiently below high water surface to provide the required capacity and the opening should be closed by a dike built to the proper elevation, as previously described. In very erodible materials, it may be possible to merely excavate a pilot channel and let the flood excavate the rest. In such cases, however, care must be taken to avoid the possibility of a side slope slide which would block the flow of water and render the spillway ineffective.

One usually associates the operation of an emergency spillway with an amount of resulting damage which, while extremely infrequent, would be considerable. However, a variation of the emergency spillway, which is associated with relatively little damage, is the construction of dikes in place of part of the gates in a multiple-gated spillway, with their tops below the elevation of the top of the dam.

These dikes serve two purposes. First, they provide an outlet to floods in the event of neglect to open the gates. Second, they are much cheaper to install than gates, and the need for their use and subsequent replacement is too remote for economic consideration.

Where several such dikes are used, they are usually built to slightly different elevations, in order that they will not all fail at once, the theory being that the failure of one or possibly two might be sufficient to pass the flood. Provision must of course be made that they are not subjected to wave action, for reasons previously explained.

Another variation consists in substituting stop logs or flashboards in place of gates, the supports of which can be tripped. (See Art. 4*b*, Chapter 24.) This and the first-mentioned variation are sometimes provided with no channel improvement below the emergency portion of the spillway so that considerable scour would occur if the spillway should operate.

INDEX